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Bureau of Mines Information Circular/1977



# Design Guide for Metal and Nonmetal Tailings Disposal

UNITED STATES DEPARTMENT OF THE INTERIOR

Information Circular 8755

# Design Guide for Metal and Nonmetal Tailings Disposal

By Roy L. Soderberg and Richard A. Busch

UNITED STATES DEPARTMENT OF THE INTERIOR
Cecil D. Andrus, Secretary

BUREAU OF MINES

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. administration.



This publication has been cataloged as follows:

Soderberg, Roy L

Design guide for metal and nonmetal tailings disposal / by Roy L. Soderberg and Richard A. Busch. [Washington] : United States Department of the Interior, Bureau of Mines, 1977.

136 p. ; 27 cm. (Information circular - Bureau of Mines ; 8755)

Bibliography: p. 107-111.

1. Tailings (Metallurgy). 2. Mineral industries - Waste disposal. I. Busch, Richard A., joint author. II. United States. Bureau of Mines. III. Title. IV. Series: United States. Bureau of Mines. Information circular - Bureau of Mines ; 8755.

TN23.U71 no. 8755 622.06173

U.S. Dept. of the Int. Library

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Washington, D.C. 20402

Stock No. 024-004-01915-8

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# DESIGN GUIDE FOR METAL AND NONMETAL TAILINGS DISPOSAL

by

Roy L. Soderberg[1] and Richard A. Busch[2]

ABSTRACT

The Bureau of Mines has conducted substantial research on the design, construction, and operation of metal and nonmetal tailings ponds. This design guide, like related Bureau publications that preceded it, is produced to assist the industry in the management of mill tailings disposal. It covers the site selection, sampling, laboratory testing, design, construction, operation, and inspection of tailings embankments. The effects of environment, topography, and hydrogeology are also included, and various methods of stability analysis and factors affecting stability are reviewed. Because of the diversity of problems encountered in tailings embankments, specific solutions are not intended. The guide is, however, a useful checklist for designers, operators, and inspectors of this type of structure.

## INTRODUCTION

This tailings disposal design guide has been prepared by the Bureau of Mines especially for those mining engineers and Government officials responsible for the design, construction, operation, and inspection of mine tailings ponds. These design recommendations are intended to deal specifically with waste from metal and nonmetal ores; however, many of the points discussed are applicable to coal waste embankments, dry mine waste piles, leach dumps, and strip and placer operations which are not specifically covered in this report.

The mining and processing of low-grade metallic ores results in large quantities of waste which leave the plant as a slurry with a 30- to 50-percent pulp density containing as much as 30 to 80 percent material of minus 200-mesh size. This slurry is retained in the tailings ponds, allowing the solids to settle out. The decant water may be recycled or allowed to discharge into a watercourse. Mining operations of 30,000 to 100,000 tons per day are not uncommon with 95+ percent being waste which has to be stored in tailings ponds. The size of these ponds has increased tremendously in the last 10 years; for

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example, in 1938, 1 ton of ore produced 27 pounds of copper; 1947--18 pounds; 1960--14.4 pounds; and 1971--11 pounds. This trend will probably continue, but at a reduced rate. The disposal problem will get worse in the future as larger tonnages are milled and land becomes more costly. The height of the dams will have to be increased, compounding the stability problems.

Research has been conducted on tailings embankments in mountainous areas where long winters with snow and freezing weather are major obstacles and in desert areas where the main problems are seepage into the ground water, dust, and water conservation.

Previous Bureau publications have dealt with various phases of tailings disposal, including design and operation, stability analysis, and seepage (30-33).[3] This report presents additional information on these subjects as well as the newest techniques for reducing seepage into the ground water and for improving the stability and safety of the embankments.

## ACKNOWLEDGMENTS

The authors are grateful to American Smelting and Refining Co. (ASARCO) for allowing research to be conducted on various properties and for its assistance in this work, and to other U.S. mining companies for allowing Bureau personnel to inspect and study their operations. We are also indebted to the following personnel of the Bureau's Spokane Mining Research Center: C. D. Kealy and M. McDonald for technical assistance on soil mechanics and computer output, and L. Atkins, R. Carnes, and D. McKenzie for the laboratory test work.

## BACKGROUND INFORMATION

The prime purpose of the design guide is to outline the criteria relating to waste disposal, some of the problems that will be encountered, and how to solve them. The guide includes explicit details of site investigation, design requirements and specifications, construction techniques, inspection procedures, and detailed investigations including sampling and testing to check the stability of present and future embankments by use of the computer. It is imperative that the design and operation of waste sites be conducted under the direct supervision of engineers who are competent in the fields of waste disposal, construction, soil mechanics, geology, hydrology, and hydraulics.

### Tailings Ponds

As used in this guide, tailings ponds comprise embankments placed on the ground surface that are required to retain slurries of waste and water; they are constructed from tailings, borrow material, or some of each. Some mines use deslimed tailings for underground fill, leaving only the finer material to be impounded on the surface. The materials range from chemically stable quartz to unstable feldspars which can alter to micrometer-size clay.

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[3] Underlined numbers in parentheses refer to items in the bibliography preceding the appendixes.

An adequate or satisfactory tailings embankment is defined as one that has a good factor of safety, will retain solids, and will control the liquid waste. Prevention of pollution by both solids and liquid must be incorporated in the design plans, together with shapes and stable slopes that will enhance rehabilitation of the area after it has been abandoned.

### Function of Tailings Ponds

The main function of a mine tailings pond is to store solids permanently and to retain water temporarily. The length of time that water must be retained ranges from a few days to months, depending on gradation, mineralogy, etc. When clarified, the water can be reclaimed for plant use or discharged into the drainage.

When the water contains a serious pollutant, the tailings dam must be designed to retain the water for longer periods until the harmful chemicals have degraded or until the water evaporates. A completely closed system is preferred in all such cases, not only for conservation of water, but as a necessity to prevent the pollutant from being discharged. The seepage water from this type of dam must be controlled, treated, and pumped back to the mill for reuse.

## BASIC CONSIDERATIONS

Economics continue to be of prime importance in the design of tailings embankments, including site selection, pumping requirements, length of pipe line, and capital versus operating cost. The annual tonnage versus site acreage, physical properties of tailings, type of embankment, method of waste disposal, availability of construction materials, climate, terrain, hydrology, geology, and nature of the foundation at alternative sites are all important factors. The consequences of failure should be fully considered in establishing the factor of safety (FS) of the embankment design. Embankments in seismically active areas should undergo dynamic analysis to eliminate the possibility of liquefaction from earthquake shock. Embankments in remote areas can have a lower FS than needed in urban areas. Operating costs for tailings disposal can be a big item in a mining operation, and much thought should go into the study of capital versus operating cost. In some cases, the plan with the cheapest capital cost can be the most expensive when the operating cost is added, and vice versa. Probably the cheapest operation possible would be one where a few water-type dams could be constructed to enclose a large area, allowing the operator to merely dump the tailings; this would completely eliminate operating labor except for pump operation and periodic inspections.

### Daily Tonnage

Operation of porphyry copper, taconite, and pebble phosphate mines can more easily anticipate the ultimate area needed for tailings disposal for the life of the deposit than can operation of underground deep-vein mines. These surface deposits are generally well defined with known ore reserves for a given number of years. Knowing this and the anticipated daily tonnage, definite plans for a tailings disposal area can be made. Any planned expansion should be considered at the same time, keeping approximately 35 acres per

1,000 tons of mill production for metal mines, preferably in two separate areas. Taconite operations require about the same acreage per 1,000 tons of waste produced. Under special conditions, such as single-point discharge into large areas where cheap land is available and other factors are favorable, the area per 1,000 tons of waste could go up to two to three times this, but observation of well-engineered taconite tailings areas indicates that 35 acres per 1,000 tons is about optimum where discharge pipelines surround the area. Phosphate mines in flat terrain will require nearly an acre of settling pond per acre of mined land until some improvement in settling rate can be achieved. Because of the fineness of the material and the low pulp density, it is generally deposited at a single point at a time.

## Size of Tailings Area

The size of the tailings embankment necessary for each 1,000 tons of milling capacity for a safe and efficient operation is governed to some extent by the size of the grind, but mostly by the terrain within the tailings area. A relatively level area of a wide, open valley is an ideal site because of the large volume of tailings placed per foot of elevation rise. A starter dam constructed from borrow material is a very important part of the entire impoundment. The purpose of this dam is to contain the sand and provide a pond large enough to insure sufficient water clarification at the start of operations. The steeper the terrain within the embankment area, the higher the starter dam must be to supply the storage necessary for the sand and water until the embankment can be raised with the beach sand. It is far better to make the starter dam a bit higher than required because of the unknown factors at startup of an impoundment. These unknowns are (1) the efficiency of segregation of the sand and slime on the beach, (2) the angle of the beach area, and (3) most important, the retention time in the pond to get clean water. A capacity curve plotting the volume against elevation should be made, as well as a time-capacity curve to get the elevation rise per year through the life of the impoundment (fig. 1).

Where the maximum annual rise is limited to less than 8 feet per year, the active disposal area must be at least 20 acres per 1,000 tons of daily capacity. Operating at this upper limit of rise per year for continuous operation might be safe, but this depends on the grind, pulp density, and type of material being impounded. From an operating and safety point of view, a figure of 30 acres per 1,000 tons of daily capacity is much better for the lower limit of a mature pond. If the site is on a hillside, the startup time is most critical because the area of active storage is small. There is no established rate that an embankment can be raised, but for a given material, gradation, and pulp density there is a definite maximum rate of rise above which stability becomes a problem. If the tailings cannot drain as fast as they are placed in the pond, the phreatic surface rises and comes out the face above the toe dam. When this occurs, seepage and piping take place, lowering the safety factor to the danger point. Possible solutions are to allow time for drainage and to place a filter and rock surcharge on the toe. A rapid annual rise is undesirable because the material does not have time to properly drain, consolidate, and stabilize, nor is there time to raise the peripheral dam.

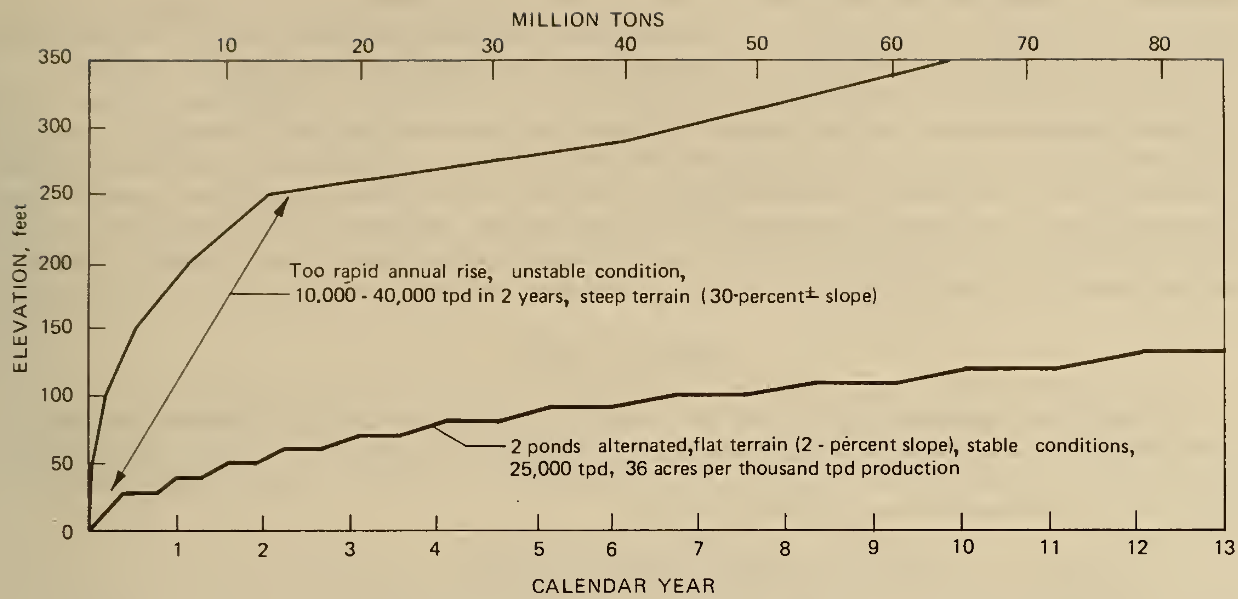


FIGURE 1. - Capacity-elevation-time curve.

The pond area required for clarification of the water prior to reclamation of discharge into the local drainage is difficult to determine by experimental or theoretical means. The problem is to provide sufficient retention time to permit the very fine fractions to settle before they reach the decant. The settlement velocities of various grain sizes and shapes can be determined theoretically; however, several factors determine the effectiveness of settlement in the field, such as grain size, percentage of slimes, pH of the water, wave action, and depth of water.

The size of grind required to liberate the metal from the waste can produce a material having 55 percent or more minus 200 mesh so that the settling rate is quite slow. Particles of 50-micrometer size have a settlement rate of 0.05 inch per second and will settle in a reasonable time even though affected by wave action. The most difficult particles to settle are those of 2 micrometers or less; these have a theoretical settlement rate of 0.01 inch per second in still water, but in fact may take days because of wave action.

The quality of the water returned to the mill or the watershed will determine the retention time for any particular mine. The time required may be as low as 2 days and as high as 10 days, with an average of about 5.

## Mill Tailings

Metal mine tailings include materials from hard quartz to mudstone with vast differences in physical properties. Finely ground mill waste high in silica can have a high shear angle at high densities with little or no cohesion and still be very susceptible to erosion by wind and water. Materials high in feldspar may have a high shear strength when fresh, but can chemically change to clay with time, reducing the strength. Relatively minor

amounts of sulfide can oxidize to form a crust and lower the pH enough that vegetative growth is difficult or impossible without adding topsoil or altering the material in some way. High-sulfide tailings may ignite by spontaneous combustion or produce acidic runoff, iron oxide, or hydroxide, which can pollute large areas in a drainage basin. The sodium cyanide from gold ore treatment plants requires a long retention time in the tailings pond, and sometimes requires treatment with chlorine or other oxidizing agents to neutralize the cyanide to tolerable levels (<0.3 ppm) before release. The waste from uranium mining and milling can be very dangerous for many years owing to radioactive daughter products.

## Particle Sizes

The grind necessary to free the metallic minerals for flotation ranges from about 30 percent to 80 percent minus 200 mesh (fig. 2). Sand-filling operations at some mines remove the coarse sand, leaving an even finer material to be impounded in tailings ponds.

Taconite plant waste products include a float product of 3/8- to 1/2-inch size and a spiral and flotation reject containing up to 70 percent minus 325 mesh, and there is a possibility of even finer grind to 90 percent minus 325 mesh to reduce the silica content in the pellets. Not all plants have the same waste products, but all could have all or part of those listed. Some

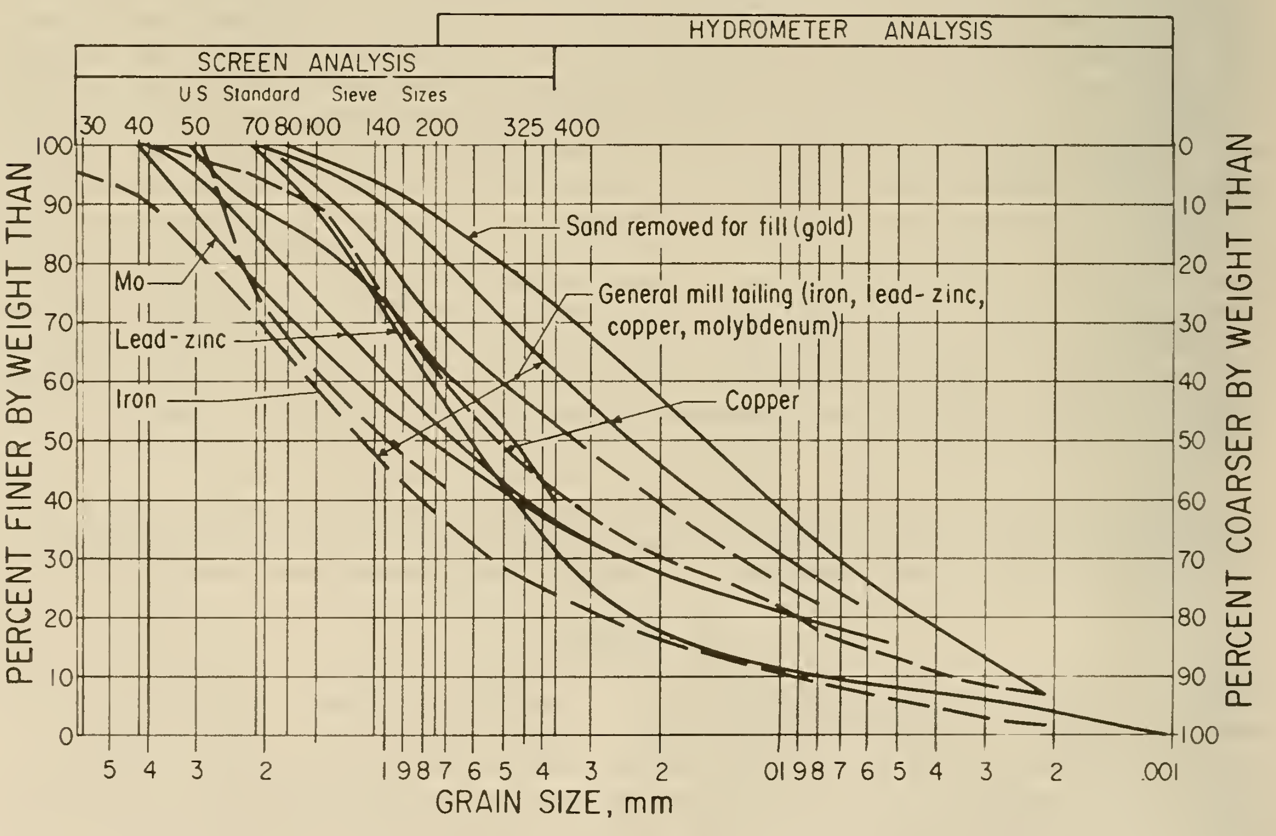


FIGURE 2. - Screen analysis–typical metal mine general mill tailings.

also have iron oxides or hydroxides that precipitate out of the seepage water, which must be contained within the impoundment.

Pebble phosphate operations have two waste products from their washing plants--a fine, white, clean silica sand similar to ocean beach sands, and a clay product all of which is smaller than 0.02 micrometer with 60 percent minus 0.001 micrometer (fig. 3). These two products are approximately two-thirds of the matrix as mined and milled. The clay waste product comes from the wash plant at 4 to 5 percent pulp density and is very slow to dewater and consolidate. For this reason it requires nearly 1 acre of pond for each acre mined.

Table 1 shows specific gravity ($G_s$), the coefficient of uniformity ($C_u$) ($D_{60}/D_{10}$), and the effective size ($D_{10}$) of some typical tailings materials. (NOTE.--$D_{60}$ is the particle size diameter (D) at "60 percent finer than" on the gradation curve.)

TABLE 1. - Physical properties of tailings

| Mineral | $G_s$ | $C_u$ | $D_{10}$, µm |
|---|---|---|---|
| Gold mine (sand removed for fill)....... | 2.6 - 2.7 | 9.20 | 2.5 |
| Iron (ore)............................ | 4.6 - 6.0 | 7.75 | 9.0 |
| Lead.................................. | 2.8 - 3.4 | 9.38 | 8.0 |
| Zinc.................................. | 2.92 | 70.00 | 3.0 |
| Copper................................ | 2.71-2.75 | 18.92 | 4.0 |
| Molybdenum............................ | 2.6 -2.7 | 20.00 | 8.0 |
| Florida phosphates....... ............ | 2.6 | 10.00 | .06 |

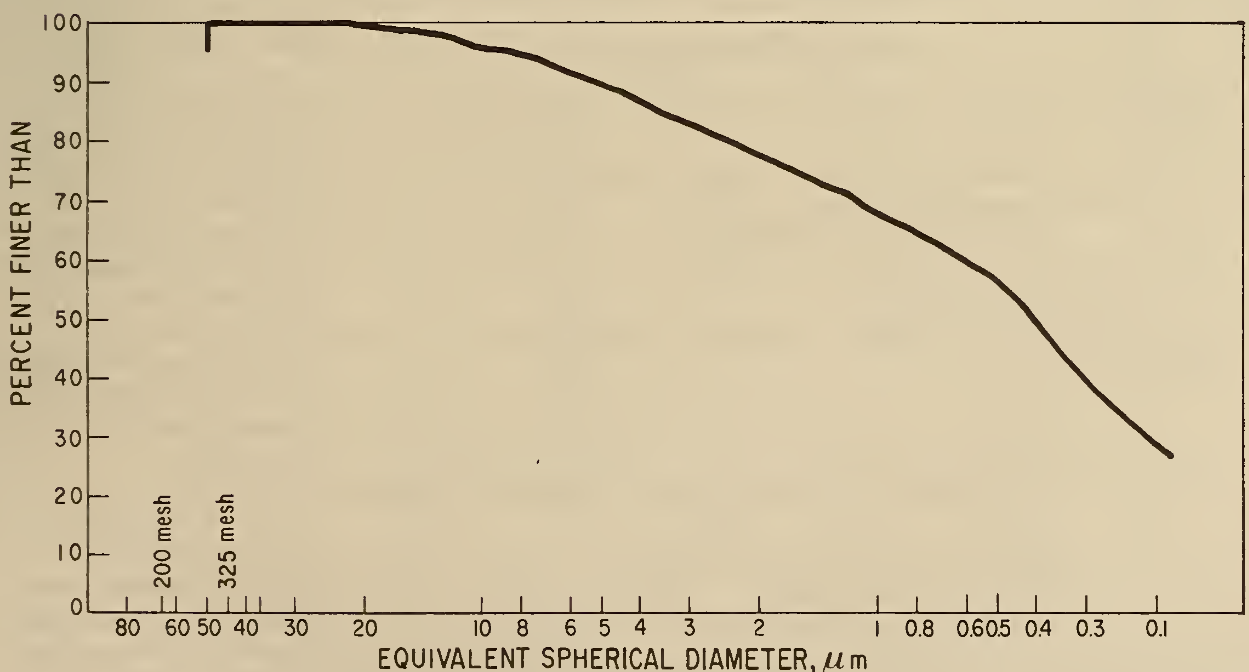


FIGURE 3. - Gradation of typical Florida phosphate slimes.

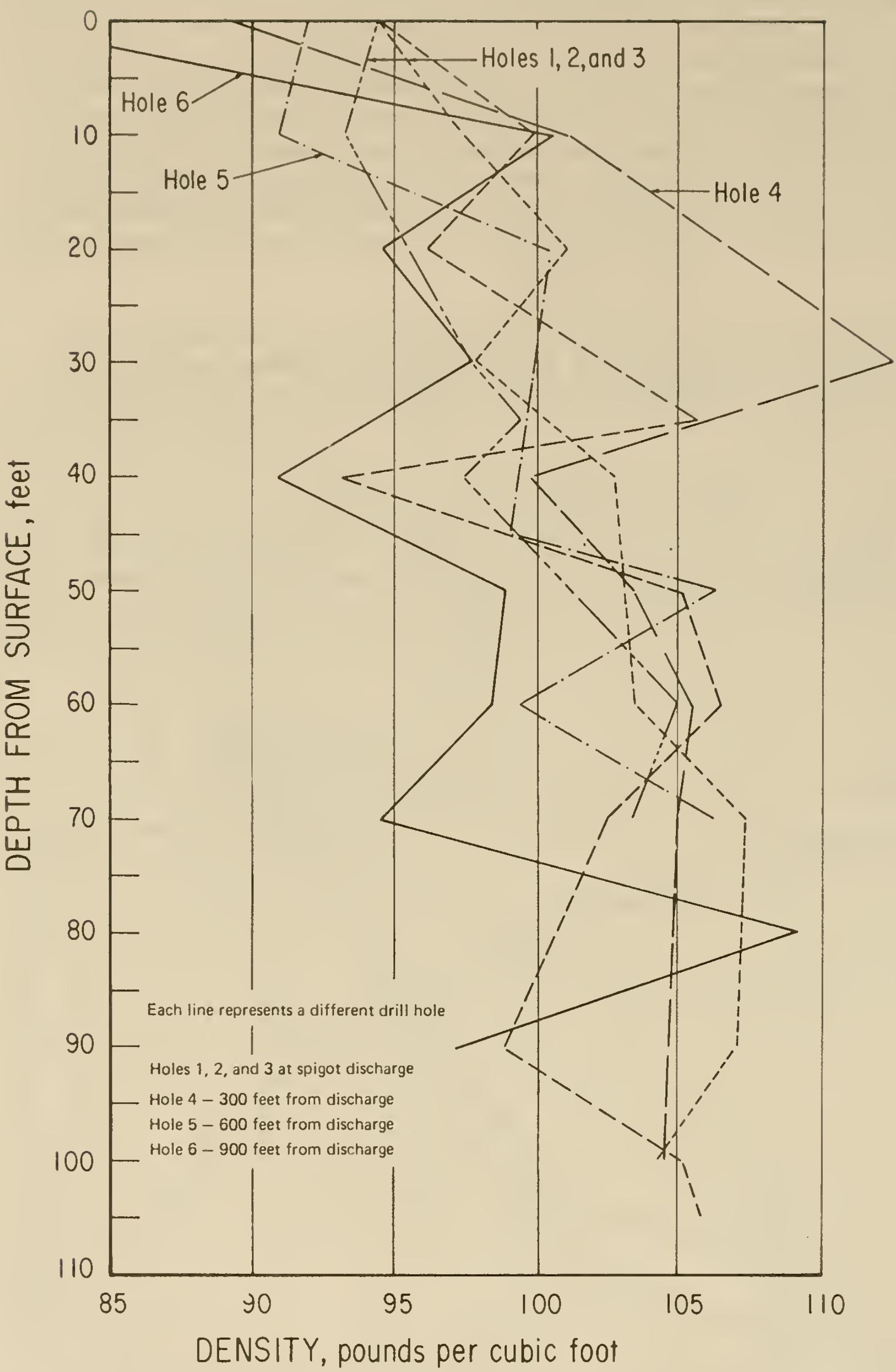


FIGURE 4. - Increased density with depth.

The field density of a tailings pond increases with time and depth below the surface. A typical example of the density change in a copper tailings pond that is in an area with a highly permeable base and where two ponds are used alternately is shown in figure 4. The density ranges from 90 to 95 pounds per cubic foot at the surface to 100 to 105 pounds per cubic foot at the 45-foot depth. In this example the inactive pond is allowed to dry so the dike can be raised for the next 10-foot fill. These tailings are discharged at 48+ percent pulp density and contains 58 percent minus 200-mesh material, resulting in a very poor segregation of coarse and fine material in the pond.

The increase in density with depth depends somewhat on the mineralogy, screen size, and specific gravity, but of more importance is the ability of the water to drain through either drains or a permeable base.

An important physical property of mill tailings is their shear strength. This property is expressed by the angle of internal friction, $\phi$, and apparent cohesion, c. Typical values for the $\phi$ angle are 20° to 35°, increasing with increased percentage of sand in the tailings. Apparent cohesion is the function of mineralogy, moisture, and particle spacing; typical values range from 0 to 5 psi.

## Mine Waste

Investigation and testing of mine waste (not tailings ponds) and mine leach dumps are generally similar to these phases of dam or highway construction. The stability of waste embankments requires the investigation of the following factors for a critical analysis:

1. Natural degradation of the waste materials due to time and weather, resulting in a loss of shear strength.

2. Water levels in the embankments. These are especially critical in leach dumps, where millions of gallons of water are pumped into the dumps each day, or when drainage water enters the embankment.

3. Sealing of embankment exteriors by weathering.

4. Incompetent material beneath the embankment such as clays, etc.

To evaluate these conditions it may be necessary to--

1. Research similar wastes in existing embankments.

2. Drill, sample, and test for physical properties.

3. Instrument the embankments to monitor continuing behavior utilizing--

   A. Piezometers and open wells to measure water pressure.

   B. Slope indicator and surface monuments for deformation measurements.

   C. Photographs that might show changes in appearance (aerial or ground photos).

   D. Monthly or quarterly surveys for a permanent record.

   E. Pumping tests to estimate water volume in dump.

Materials that commonly degrade rapidly are shales, siltstones, and mudstones; the feldspars can degrade into clay. Some feldspars found in swelling porphyry are already partly decomposed and could break down quite rapidly if excavated and exposed to the elements. Often these materials will gradually lose their strength when exposed to freezing, thawing, wetting, and drying, or to the relief of stress by excavation. If the loss of strength is substantial,

a waste pile designed based on the strength of the fresh material could become unstable at some time in the future. Therefore, piles of waste that may be susceptible to degradation should be designed on the basis of the lowest strength that the material might attain. Accelerated tests of degradation can be performed in the laboratory using rapid cycles of freezing and thawing, wetting and drying, or exposure to acids such as those used in copper leach dumps that might influence the waste. These tests may not indicate the final strength of the material but could indicate that the waste material may change with a reduction in strength. Borrow material may be required to confine the pile rather than relying on the material itself for stability. This material should always be protected so that surface drainage does not flow through it because of leaching of metallic ions, degradation, stability, etc. Samples of the degraded material may be found in the field and would be much more nearly representative when testing for strength.

Many "waste" embankments that have been accumulating for over 50 years are now being leached for the recovery of copper. No special care in placing the material to insure stability seemed necessary because the material was dry and was to remain dry during the early stages of placement. With the advent of leaching, these vast piles of "waste" became partially saturated and stability problems developed. Impervious zones at each lift allow leach solutions to cascade out onto the face, added weight causes bulging of the foundation material, and degradation due to wetting the material itself results in relatively impervious zones. These dumps can absorb billions of gallons of water before a water balance is attained. Stability then becomes a problem, and piezometers and/or open wells for monitoring the phreatic line are very important. Slope indicators or surface monuments should be installed to monitor movement of the embankment, and drilling and sampling should extend into the foundation material.

### Leach Dumps

Most U.S. leach dumps consist of waste and low-grade ore that has accumulated from mines and open pits over a period of years (up to 50 years or more); this material now can be leached at a profit by simply spraying or injecting water or dilute acid over the surface and collecting the pregnant solution for precipitation. The gradation ranges from large boulders to fine clay with a continuing change in physical properties as the material changes with time and chemical action. Stability analyses of these embankments are very difficult to make because of the heterogeneous mixture of the materials. When low-grade ores are placed into leach dumps, all roadways and hard-packed areas should be thoroughly ripped before they are covered with more ore.

## SITE SELECTION

The selection of a site for tailings disposal has to be made when the plant and mine sites are selected. In the feasibility study of a new property, a tentative tailings site must be picked. It should be within a radius of 10 miles, preferably as close to the mill as possible, and downstream from the mill for gravity flow of the tailings. It must be of adequate size to accommodate the annual tonnage of tailings without too rapid rise in the height of the embankment each year.

In a new area and early in the mine exploration period (as soon as it becomes apparent that a mine is in the making), data should be gathered in the area. All climatic data should be gathered, and onsite measurements of stream flow and evaporation should be made. Sedimentation characteristics, turbidity, pH, metallic ion count, etc., on the proposed waste should be determined. U.S. Geological Survey (USGS) topographical maps are usually available. Detailed contour maps of the impoundment area are necessary for the planning and design of mine waste embankments. Aerial photographs are useful for locating geological features that may not be discernible by surface reconnaisance and mapping and for locating potential sources of construction materials.

The USGS maps are valuable for reconnaisance surveys, for choosing a site, for measuring area and volume, and for general geology, drainage area, creeks, etc. Major faults should be avoided in the tailings area and especially in the dam area. By the time of site selection, there should be enough geological information available to eliminate potential tailings sites on any mineralized areas, vein extensions, potential shaft sites, pit access, or possible pit extensions. The site should be far enough from projected mining to preclude seepage, spills, or runs into the mine through faults, shafts, or fractures from mining operations.

Habitation downstream from a potential tailings dam would affect the design in that a higher factor of safety would be necessary than in a remote area.

## Soils and Construction Material Investigations

Site investigation for low embankments of 100 feet or less on sites where bedrock is at shallow depths can be assessed by auger holes and test pits. For plants with large daily tonnages, where land and capital costs for tailings disposal are high, design must be for large areas with up to 500-foot-high embankments. This requires careful and detailed study of the foundation materials, especially if clays or silts are present. Extensive foundation drilling, sampling, and testing may be necessary. Soil samples should be tested for inplace density, gradation, shear strength, consolidation, and moisture content. These tests are also needed for location and availability of borrow material for toe dam construction.

Organic soils are generally very compressible, have low shear strength, and should be removed from embankment foundations. When saturated or under load, they could act as a lubricant and cause a failure.

Coarse, sound angular rock, such as talus, will be strong and pervious, but settlement can be expected if it is uncompacted. Some sedimentary rocks such as shales and mudstones weather severely and reduce the shear strength of this type of fill.

Solid bedrock has more than adequate compressive and shear strength to support mine waste impoundments. Where dams are to be constructed on or near bedrock, surface springs or artesian water can be a danger. Faults or fault gouge can affect the stability of an embankment.

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The physical properties of glacial tills and similar soil mixtures depend on their densities and gradations. The amount and type of fines are important.

Water levels should be measured in all exploratory holes and test pits, especially where materials of different permeability are encountered. Artesian water should be noted and considered in the design.

Pneumatic piezometers should be placed in strategic drill holes in the foundation and carefully mapped for location and elevation to provide continuous reference during the life of the embankment.

Most tailings embankments require a toe dam or starter dam. If the tailings themselves are to be used for embankment construction, the specific gravity, gradation, permeability, and shear strengths at the expected density of the tailings are necessary. Given the geometry, density, shear strength, and anticipated phreatic surface, a stability analysis should be run on the embankment to check the factor of safety at its designed height, say 500 feet. If the FS is too low, additional compaction or a flatter downstream slope may be necessary to bring the FS to a satisfactory figure.

## Topography and Geology

Topographic maps necessary for planning a mine waste embankment can be obtained from the USGS. These topographic maps are available in various scales: 1:125,000 at 100-foot contour intervals; 1:62,500 at 50-foot and 40-foot intervals; 1:24,000 at 40-, 20-, and 10-foot intervals; and 1:12,000 at 40- and 20-foot intervals. When an area has been chosen, more detailed topographic mapping may have to be done locally, especially where the toe dam and drains are to be built.

Aerial photos of most of the United States are available from USGS in Menlo Park, Calif. They are a help in geological mapping because faults, different types of rock, ground cover, etc., are noticeable. Local detailed geology will probably have to be done by the company building the embankment or by a consultant hired to do this work. It is essential that this be done well and in great detail to be sure there are no weak or incompetent soil or rock, no major faults, and no ore deposits in the immediate area.

The extent of geological investigation necessary for a tailings impoundment will depend on the height to which it is to be built and the complexity of the foundation material. The foundation must be firm enough to prevent undue settlement, strong enough to withstand the shear stresses, and of a nature that seepage can be controlled.

For a major tailings impoundment a logical sequence of geological investigation should include:

1. Location and study of geological reports, maps, and photographs.

2. Field reconnaissance, including surveying and mapping of surface deposits, their extent and mode of occurrence, any outcrops, etc.

3. If overburden is deep, geophysical surveys may be necessary to determine the depth.

4. Measurement of ground water levels, which may also include pumping tests.

5. Location of all seeps and springs within the tailings area and especially in the dam area.

6. Core drilling for location of faults, planes of weakness, mineralization, and ground water. The main reason for core drilling is to check for mineralization. Any other information is a bonus and may be very helpful.

7. Evidence of solution cavities or collapse of cover, which are common in limestone beds.

8. Evidence of sloughing along valley walls caused by water, clay seams, or weak formations, indicating unstable conditions.

The geological history of the surface deposits in and near the embankment site can often dictate the design and construction of the initial dam. The origin of the deposits and whether they have been subjected to consolidation pressures since their formation will indicate the physical properties that might be expected. Highly compacted and consolidated soils which demonstrate good shear strength are generally ample foundation for an embankment. Bedrock makes an excellent foundation, provided there are no extensive soft seams, or the material is not soft weathered shale, mudstone, schist, etc. If the overburden is shallow, the bedrock essentially becomes the foundation, and under these conditions it should have more than ample strength.

## Climate and Hydrology

When exploration is being carried on for a new property in relatively remote and virgin country, hydrologic investigations should be started. The National Weather Service has records available for the entire United States, but if the area is remote from a weather station, site records should be made of daily temperature, precipitation, solar radiation, wind, and evaporation. The Water Resources Division of the USGS has streamflow measurements of most major streams, but here again specific minor tributary streams should be measured. Water quality of all the watersheds in the vicinity should be checked for turbidity, metallic ions, pH, etc. The latter is especially important in that a stream flowing from a heavily mineralized area might have a higher metallic ion content in its natural state than is allowed in the State and Federal regulations. A 3- to 4-year history of streamflow beside, through, or beneath a tailings impoundment is important for the designer. The probability of a 100-year flood, or larger, is also important.

## Evaporation

In the arid Southwest as much as 84 inches of water per year may be lost by evaporation from a tailings pond, and this is one of the major water losses.

Evaporation from water surface is influenced by solar radiation, wind, air temperature, and vapor pressure. Since solar radiation is an important factor, evaporation varies with latitude, season, time of day, and cloud cover.

The National Weather Service Class A pan is used for estimating evaporation from lakes and reservoirs. It consists of an unpainted galvanized iron pan, 4 feet in diameter and 8 inches deep, placed on a wood frame to allow air to circulate completely around it. It is filled daily to a depth of 8 inches for 12 to 18 months, and evaporation is monitored. Additional instruments can be installed near the evaporating pan to relate the measured evaporation in the pan to meteorological factors. Some of these instruments are--

1. Wet- and dry-bulb thermometers for air and precipitation temperatures, vapor pressure, and dew points.

2. Anemometer for wind.

3. Precipitation gages--one nonrecording and one weighing-type recording gage.

Pan coefficients (ratio of lake evaporation to pan evaporation) are used to estimate the evaporation from lakes and reservoirs. The evaporation from natural lakes and reservoirs is 0.6 to 0.8 as much as from the Class A pan, a coefficient of 0.7 is a good average figure.

Appendix A gives further information on evaporation.

## Runoff

Runoff must be considered in designing a mine tailings pond. The annual spring runoff can best be assessed by even a few years of records for a given watershed. Where this information is not available and the watershed is small, hydraulic handbooks have simple equations to calculate runoff flow rates. The National Weather Service has maximum probable precipitation for a general area which can be used, and assuming a saturated watershed, a runoff hydrograph can be drawn. The design must be made to handle the 100-year flood whether it is by spillway, diversion ditch, decant tower with discharge lines, or pipe beneath the embankment.

In areas of high snowfall the maximum rain could occur in the winter on a deep snow pack with above-freezing temperatures. The runoff could be increased by melting of a large portion of the snow, so that the total runoff could be even greater than the total rainfall.

A reliable method for estimating runoff volume and flow requires three steps. Step 1 is the estimation of the amount of precipitation in the form of rain or snow for a duration equal to the time of concentration for the area. This information is available from the National Weather Service, as are the maximum storm and the probable frequency of occurrences. Step 2 is the assessment of the runoff losses in the catchment area by vegetation,

evaporation, infiltration, and storage in lakes, etc., all depending on the characteristics of the area. This step can be eliminated by being on the safe side and assuming a saturated watershed, which often happens when the main storm is preceded by many days of rain.

Step 3 then assumes that all the precipitation is runoff and the timing and quantity of the maximum flow are the only problems. A few years' record of precipitation and streamflow in that drainage will show the shape of the hydrograph, which should be more accurate than a synthetic streamflow hydrograph. (Synthetic hydrographs are drawn from generalized data available on published climatic maps and records from adjacent areas.) To obtain onsite information recording and nonrecording rain gages, a snow storage gage, and a recording streamflow measurement gage are necessary. Streamflow measurements are also required to determine the stage-discharge relationship of the stream gage.

The National Weather Service has records of precipitation, and the Water Resources Division of the USGS has streamflow records and hydrographs which can supply information for a specific watershed not directly covered by their streamflow gages.

## SUBSURFACE EXPLORATION

Two main goals of subsurface investigations are to find evidence of previous mining operations and to determine the possibility of future mining under the dam area since such mines may collapse due to the additional loading caused by the impoundment. Caving or sublevel stoping without fill can cause fracturing far from the actual mine and allow tailings from a superimposed tailings pond to run into the mine.

Surface trenching and test pits are the most economical method of sampling to obtain the physical properties, quantities, and quality of materials for dam building, drainage, filters, etc. Drilling may be necessary in the deeper overburden using such drilling equipment as a wash boring machine with casing, churn drill, rotary drill using mud, diamond drill, hammer drill using casing with air, or Becker[4] hammer drill without casing using air. Shelby tube samples, split-spoon samples for penetration resistance, and disturbed samples may be obtained from the drill holes. The Becker drill delivers a continuous disturbed sample but is limited where large hard boulders are encountered. Power augers with hollow stem and continuous flight augers allow sampling through the hollow stem without casing but are also limited to small gravel and finer soils.

If the subsoil is very competent and uniform, a minimum amount of drilling will be required, but if it is erratic with seams of clay, soft sand, or many dissimilar soils, enough drilling should be done to obtain a soil profile.

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[4]Reference to specific trade names does not imply endorsement by the Bureau of Mines.

The foundation borings should be deep enough to determine the subsoil characteristics within the depth affected by the structure. A competent foundation material is not difficult to attain in most instances, but there are problem areas that should be mentioned including buried talus deposits that are very pervious, solution cavities in limestone, and soft seams in an otherwise competent clay. Another problem is deep overburden where nearly all the seepage goes out the bottom of the pond and not out the downstream face where it can be reused or monitored for quality before going into the drainage. This will become increasingly important in the future when seepage into the groundwater will not be tolerated. Shallow overburden with an impervious bedrock is good because a trench can be cut into the bedrock to catch and control the seepage.

## Logging Test Holes

Typical forms for logging test hole or pit samples are shown in figure 5; figure 6 is a key for exploration logs. Different observers should try their best to standardize their interpretation of various materials encountered and tested. Logging of samples should be done by a trained soils engineer, who should be on the job continuously during sampling to keep records up to date, and the logs should be prepared in final form as soon as possible after the hole is completed. Hvorslev (28) treats this subject in considerable detail.

Complete and accurate detail of the samples taken from test pits and drill holes is critical for correct analysis of an embankment site. Logs should be kept of the elevations and thicknesses of the various strata or horizons that are encountered. In addition, the color, structure, estimated textural classification, and any other identifying characteristics of the soil in each horizon should be noted. The designation of the textural class of a soil in the field is only a temporary identification procedure which will be modified or made permanent after the results of a laboratory analysis become available. However, the personnel of a soil survey party may become quite proficient in estimating the textural classification by visual examination in the field.

Drilling and sampling for site investigation should also include the location of burrow material for dam construction. Sampling should follow the same pattern as for foundations and be extensive enough to assure an ample supply of the needed material as close to the dam site as possible.

| DRILL RIG FAILING 1500 | HOLE ELEVATION, 3,812.9 feet | LOGGED BY LA |
|---|---|---|
| GROUND WATER DEPTH NOT DETERMINED COOLED GROUND SURFACE | HOLE DIAMETER 4–7/8 inches | DATE DRILLED DEC. 11, 1971 |
| Hole located on lower berm of Dam 2W. | | |

| ELEV., feet | CLASS | DESCRIPTION FIELD IDENTIFICATION | SAMPLE NUMBER | MODE | REMARKS |
|---|---|---|---|---|---|
| 75 | | | | | This hole represents continuation of hole MD–2 where circulation of drilling fluid was lost at 11 feet after reaching 83.5 feet. Hole set 29 feet from MD–2 |
| 80 | | | | | |
| 85 | SM to ML | 0 to 124.0 SAND, very silty; gray brown, with sandy SILT lenses | | RD | |
| 90 | | | S–12 89.0–91.0 | S | 220 psi |
| | | | B–5 91.0–92.5 | DR | 7/.5,11/.5,16/.5 N=27 |
| | | | | RD | |
| 95 | | | | | 420 psi |
| | | | S–13 97.0–100.0 | OSTERBERG | |
| 100 | | | B–6 100.0–101.5 | DR | 7/.5,13/.5,17/.5 N=30 |
| | | | | RO | |
| 105 | | | | | |
| | | | S–14 107.0–109.0 | S | 240 psi |
| 110 | | | | DR | 11/.5,10/.5,16/.5 N=26 |
| | | | | RO | Hole started squeezing |
| 115 | | | | | |
| | | | S–15 118.0–117.3 | S | 400 psi Refusal due to full Shelby tube. |
| | | | B–7 117.3–118.8 | DR | 15/.5,11/.5,15/.5 N=26 |
| 120 | | | | RO | |
| 125 | | 124.0 to 134.0 feet BEDROCK: GILA CONGLOMERATE | | | Bedrock at 124 feet |

N = Blow count for penetration of split - spoon sampler (last foot)

| DRILL RIG FAILING 1500 | HOLE ELEVATION, 3,812.9 feet | LOGGED BY LA |
|---|---|---|
| GROUND WATER DEPTH NOT DETERMINED COOLED GROUND SURFACE | HOLE DIAMETER 4–7/8 inches | DATE DRILLED DEC. 11, 13, 1971 |

| ELEV. feet | CLASS | DESCRIPTION FIELD IDENTIFICATION | SAMPLE NUMBER | MODE | REMARKS |
|---|---|---|---|---|---|
| 125 | | BEDROCK: GILA CONGLOMERATE – Continued | | RO | Moderately difficult drilling with 120 psi applied pressure. No fluid loss. |
| 130 | | | | | |
| 135 | | BOTTOM OF HOLE – 134. Set piezometer tip at 130.0 feet. Filled hole with sand–pea–gravel to 112.0 feet. Hole started to cave. Set 5 feet bentonite plug from 107.0 feet to 112.0 feet and backfilled hole with cuttings. | | | Hole stopped at 134 feet 10 feet into Gila Conglomerate. Hole washed out with clear water to install piezometer |

N = Blow count for penetration of split - spoon sampler (last foot)

FIGURE 5. - Typical log of exploration hole (51).

| UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487) | | | | |
|---|---|---|---|---|
| MAJOR DIVISIONS | | | GROUP SYMBOLS | TYPICAL NAMES |
| COARSE-GRAINED SOILS More than 50 pct retained on 200 sieve* | GRAVELS 50 pct or more of coarse fraction retained on 4 sieve | Clean gravels | GW | Well-graded gravels and gravel-sand mixtures, little or no fines. |
| | | | GP | Poorly graded gravels and gravel-sand mixtures, little or no fines. |
| | | Gravel with fines | GM | Silty gravels, gravel-sand-silt mixtures. |
| | | | GC | Clayey gravels, gravel-sand-clay mixtures. |
| | SANDS More than 50 pct of coarse fraction passes 4 sieve | Clean sands | SW | Well-graded sands and gravelly sands, little or no fines. |
| | | | SP | Poorly graded sands and gravelly sands, little or no fines |
| | | Sands with fines | SM | Silty sands, sand-silt mixtures. |
| | | | SC | Clayey sands, sand-clay mixtures. |
| FINE-GRAINED SOILS 50 pct or more passes 200 sieve* | SILTS AND CLAYS | Liquid limit 50 pct less | ML | Inorganic silts, very fine sands, rock flour, silty or clayey fine sands. |
| | | | CL | Inorganic clays of low to medium plasticity, gravelly clays. Sandy clays, silty clays, lean clays. |
| | | | OL | Organic silts and organic silty clays of low plasticity. |
| | | Liquid limit greater than 50 pct | MH | Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts. |
| | | | CH | Inorganic clays of high plasticity, fat clays. |
| | | | OH | Organic clays of medium to high plasticity. |
| | Highly organic soils | | PT | Peat, muck, and other highly organic soils. |

DEFINITION OF TERMS
GRAIN SIZES

| | U.S. STANDARD SERIES SIEVES | | | | CLEAR SQUARE SIEVE OPENINGS | | |
|---|---|---|---|---|---|---|---|
| | 200 | 50 | 16 | 4 | 3/4 inch | 3 inch | 12 inch |
| Silts and clays distinguished on basis of plasticity | Sand | | | Gravel | | Cobbles | Boulders |
| | Fine | Medium | Coarse | Fine | Coarse | | |

| MOISTURE CONDITION (INCREASING MOISTURE ——) | | | | | |
|---|---|---|---|---|---|
| Dry | Slightly damp | Damp | Moist<br>Plastic limit | Very moist | Wet (saturated)<br>Liquid limit |

| SAMPLE NUMBER COLUMN | | MODE COLUMN | | REMARKS COLUMN |
|---|---|---|---|---|
| Type of sample container: | | Method of advancing hole: | | Number of blows required to drive sampler is shown for each 0.5 foot of penetration as follows: 17/.5 22/.5 29/.5 |
| Bag | B | Drill: | | |
| | | Flight auger | AD | |
| Jar | J | Bucket auger | BA | |
| | | Spin auger | SD | N = number of blows required to drive sampler the last 1 foot |
| Shelby tube | S | Rotary drill | RD | |
| | | Cable tool | CT | Terminated hole: sufficient information obtained. |
| Liner (tube) | L | Penetrometer sampler: | | |
| Wrapped core | WC | Drive | DR | Refusal: stopped by material too hard for equipment. |
| Box | X | Pitcher barrel | PB | |
| | | Core | C | Abandoned hole: Stopped because of difficulties as explained on log. |
| Pitcher tube | PB | Recovery ratio indicated by a fraction: $\frac{1.2}{1.5} = \frac{\text{Footage recovered}}{\text{Footage sampled}}$ | | |

| SANDS AND GRAVELS | |
|---|---|
| Relative density | Blows/foot* |
| Very loose | 0–4 |
| Loose | 4–10 |
| Medium | 10–30 |
| Dense | 30–50 |
| Very dense | Over 50 |

| CLAYS AND SILTS | | |
|---|---|---|
| Consistency | Blows/foot** | Strength*** |
| Very soft | 0–2 | 0–1/4 |
| Soft | 2–4 | 1/4–1/2 |
| Firm | 4–8 | 1/2–1 |
| Stiff | 8–16 | 1–2 |
| Very stiff | 16–32 | 2–4 |
| Hard | Over 32 | Over 4 |

*Based on the material passing the 3-inch (75-mm) sieve
**Number of blows of 140-pound hammer falling 30 inches to drive a 2-inch-OD (1-3/8-inch-ID) split spoon (spt)
***Unconfined compressive strength in tons/sq ft

FIGURE 6. - Key for exploration logs (51).

## Sampling

Samples are classified according to sampling procedures used: wash samples, disturbed samples, and undisturbed samples. Table 2 shows the sampling methods for various soil types.

TABLE 2. - Sampling methods for various soil types

| Type of soil | Methods of boring (methods shown in parentheses are rarely used) | Reconnaissance exploration, representative samples (sampling in borings of each significant stratum, 5-foot maximum spacing) | Detailed exploration, small undisturbed samples (sampling in borings, continuous samples, 2- to 3-inch diameter) | Special exploration, large undisturbed samples (sampling in borings of controlling strata, 4- to 6-inch diameter) | Surface sampling, undisturbed samples and control samples (sampling close to surface, accessible explorations and earth structures) |
|---|---|---|---|---|---|
| Common cohesive and plastic soils. | Displacement, wash, auger continuous sampling (percussion, rotary). | Augers, 1- to 2-inch piston or open-drive sampler. | Thin-wall drive sampler open or with stationary or free piston. | Thin-wall or composite drive sampler with free or stationary piston (cut, wire, vacuum relief). | 2- to 6-inch thin-wall open-drive or free-piston sampler, 4- to 8-inch advance trim. 8- to 12-inch-square box sample. |
| Slightly cohesive and brittle soils including silt, loose sand above ground water. | As above, but keep boring dry for undisturbed sampling above ground water. | .....do............... | .....do.................. | Thin-wall drive sampler, free or stationary piston (vacuum relief). | As above, but advance trimming or box sampling preferable. |
| Very soft and sticky soils. | Displacement, wash bailers, sand pumps, continuous sampling (auger, rotary). | Slit or cup sampler, 1- to 2-inch piston or open-drive sampler (core retainers used). | Thin-wall drive sampler with stationary piston. | Thin-wall or composite drive sampler with stationary piston, vacuum relief required. | 2- to 6-inch thin-wall open-drive or stationary piston sampler, danger of soil movements and disturbance before sampling. |
| Saturated silt and loose sand. | Displacement, wash bailers, sand pumps, continuous sampling (rotary). | As above, release stationary piston before any intentional overdriving. | Thin-wall drive sampler, free or stationary piston, 2-inch diameter. | Thin-wall drive sampler, free or stationary piston, vacuum relief or freezing bottom of sample required. | 2- to 6-inch thin-wall sampler, open or free or stationary piston, 4- to 8-inch advance, trim sample, depress ground water level. |
| Compact or stiff and brittle soils including dense sand, partially dried soils. | Wash, augers, percussion, rotary continuous sampling. | Augers and 1- to 2-inch thick-wall piston or open-drive sampler. | Medium-wall open-drive or piston sampler. Hammering may be required (partial disturbance). | Core boring may be better than drive sampling, but danger of contamination in partially dry soils. | 4- to 8-inch advance trim. 8- to 12-inch-square box of block samples. Auger core boring. Bag sample and field density. |

TABLE 2. - Sampling methods for various soil types--Continued

| Type of soil | Methods of boring (methods shown in parentheses are rarely used) | Reconnaissance exploration, representative samples (sampling in borings of each significant stratum, 5-foot maximum spacing) | Detailed exploration, small undisturbed samples (sampling in borings, continuous samples, 2- to 3-inch diameter) | Special exploration, large undisturbed samples (sampling in borings of controlling strata, 4- to 6-inch diameter) | Surface sampling, undisturbed samples and control samples (sampling close to surface, accessible explorations and earth structures) |
|---|---|---|---|---|---|
| Hard, highly compacted or partially cemented soils, no gravel or stones. | Percussion, rotary continuous sampling. | Thick-wall open-drive sampler. Core boring. | Thick-wall open-drive or piston sampler. Core boring. Small-diameter samples, often partially disturbed. | Core boring preferable to drive sampling. Danger of fluid contamination in permeable soils. | 8- to 12-inch-square box samples or irregular block samples. |
| Coarse gravelly and stony soils including compact and coarse glacial till. | Percussion barrel auger, loosen by explosives, thick-wall drive sampler. | Barrel auger, thick-wall drive sampler (core retainer). | Not practicable.......... | Advance freezing, then core boring. | 8- to 12-inch-square box samples, bag sample, and field density. |
| Gaseous or expanding soils (organic soft clay, silt, sand). | According to soil, but keep boring filled with water or drilling fluid. | As above, according to basic soil type. | Thin-wall sampler with free or stationary piston. Force closed sampler through expanded soil. Determine original sample length and volume. Sealing to prevent expansion. | Thin-wall sampler with free or stationary piston. Force closed sampler through expanded soil. Determine original sample length and volume. Sealing to prevent expansion. | Thin-wall drive sampler, open or piston type. Danger of expansion of soil before sampling. |
| Gradual or sudden changes in soil properties within a single drive. | As above, according to basic soil type. | .....do.............. | Safe length of sample increased when progressing from weak to firm strata and vice versa. Thin soft strata, often disturbed. Withdraw after passing firm stratum. | Safe length of sample increased when progressing from weak to firm strata and vice versa. Thin soft strata, often disturbed. Withdraw after passing firm stratum. | As above, according to soil type. When possible, separate coarse- and fine-grained soil. |
| Soils with secondary structure. | .....do........... | .....do.............. | As above, according to basic soil type, but the results of strength, consolidation, and permeability tests do not always represent properties of undisturbed deposit. | As above, according to basic soil type, but the results of strength, consolidation, and permeability tests do not always represent properties of undisturbed deposit. | Large box or block samples. Large test specimens. Detail field tests and observations. |

Wash samples consist of drill cuttings removed from the hole by circulating air or water, or simply wash sand and gravel removed ahead of casing driven into the soil or by bailing from a churn drill. The approximate stratigraphy and a preliminary soil classification can be determined from wash samples, but they are not reliable for laboratory testing or positive soil identification.

Disturbed samples are obtained in thick-walled sample tubes such as the split-spoon sampler and are useful for general classification tests and soil identification as well as gradation, specific gravity, moisture content, and Atterberg limits (4-5). They are not suitable for strength tests.

Undisturbed samples require sophisticated sampling equipment such as hollow-stem augers that require no casing, and special techniques are used to preserve the sample in its natural condition. The most common sampler is the Shelby tube, which is a thin-walled steel tube approximately 3 inches in diameter and 24 or 30 inches long. It should be pushed into the soil by a steady pressure such as a hydraulic ram and not be a hammering action. It should be sealed to prevent loss of water during handling and storage and should be transported in shock-proof containers. These undisturbed samples are suitable for all laboratory tests such as triaxial and direct shear, screen size, density, moisture, specific gravity, and void ratio. The results of these tests will reveal many characteristics of the in situ soil deposit.

Sampling from a drill hole can be comparatively simple or can require a considerable degree of experience and ingenuity, depending on the type of deposit and the degree of disturbance that is acceptable. A relatively uniform deposit may be sampled at 5-foot intervals, while weaker or highly erratic zones may require continuous sampling.

In cohesive soil, clay, and sand and in some gravel, the hollow-stem auger is ideal for use with a Shelby tube, piston, or split-spoon sampler. A blow count for relative density determination can also be obtained with the split-spoon samples. It is impossible to get Shelby tube samples in saturated sand except with the cryogenic equipment that freeze only the tip with $CO_2$ to retain the sample in the tube until it can be removed from the hole. If much gravel is present, thick-wall drive samplers or barrel augers are used, which requires care and experience to obtain good samples.

Hammer drills using air and percussion are frequently used in exploring sands, gravels, and even waste dumps containing coarse rock. Samples from this drilling are valuable as indicators of the material in the hole but are not truly representative because of degradation and segregation of particles in transport up the casing. The action of the drill can indicate if the material is coarse boulders with voids or well-graded material.

## Laboratory Testing

### Grain Size Distribution

The laboratory procedures for determining grain size is described in ASTM 422-63 (American Society for Testing and Materials) (3). In this procedure, the sample is divided into a coarse fraction and a fine fraction; the coarse material is mechanically separated on several screens and washed through a 200-mesh screen. The fine fraction is tested by hydrometer or Sedigraph to establish its gradation.

If the soil's grain size is quite variable, a number of samples must be tested to get an envelope of distribution curves to form a basis for assessing some of the soil's other characteristics.

## Specific Gravity and Void Ratio

The specific gravity of soil particles is determined by laboratory test ASTM D854-58 (7).

The void ratio (e) is defined as the ratio of void volume to solid volume in a soil mass. The void volume of a soil mass is any volume not filled with particles. Therefore, the void volume plus the solid volume is equal to the total volume.

$$\text{Void ratio, } e = \frac{G\ \gamma_{\omega}\ V}{W_s} \text{ minus } 1, \qquad (1)$$

where $G$ = specific gravity of soil solids,

$\gamma_{\omega}$ = unit weight of water,

$V$ = volume of the mass,

and $W_s$ = dry weight of soil grains.

## Relative Density

The relative density is given by the relationship

$$Dr = \frac{e\ max\ -e}{e\ max\ -e\ min} \times 100, \qquad (2)$$

where $Dr$ = percent relative density,

$e\ max$ = void ratio in soil's loosest state,

$e\ min$ = void ratio in soil's densest state,

and 3 = void ratio in situ.

## Plasticity

The Atterberg limit tests (34) are used to measure the consistency of soils related to the amount of water in the system. There are four states or conditions of the soil in terms of "limits," as follows:

1. "Liquid limit," the boundary between the liquid and plastic states.

2. "Plastic limit," the boundary between the plastic and semisolid states.

3. "Shrinkage limit," the boundary between the semisolid and solid states.

4. The difference between the first two water content values is the range of water content over which the soil remains plastic and is called the plasticity index. This parameter is used in classifying soils for estimating other physical properties which have been correlated empirically with the plasticity index (Casagrande system).

## Compaction

The moisture-density relationship for compacting soil is obtained by the Standard Proctor method (6) or the Modified Proctor method (9). In the early days of compaction, when construction equipment was small and gave relatively low densities, the Standard Proctor density was the expected value to be attained in the field. As construction equipment and procedures were developed which gave higher densities, the Modified Proctor method with over 4-1/2 times the compactive effort of the Standard was adopted.

A definite relationship exists between the water content of a soil at the time of placement and the amount of compactive effort required to achieve a given density. If silts and clays are too wet or too dry, the maximum density will not be attained with a given compactive effort. The objective of the laboratory procedure is to determine the optimum water content and maximum density for the specified compaction effort.

Sands are not as moisture dependent but should be compacted either saturated or completely dry to avoid the effect of "bulking."

## Shear Strength

The shear strength of a soil may be measured by triaxial compression tests or direct shear tests. Triaxial tests measure the shear strengths under both drained and undrained conditions with the sample maintained as near field conditions as possible. Direct shear tests can define shear strengths under limited conditions of moisture and confinement.

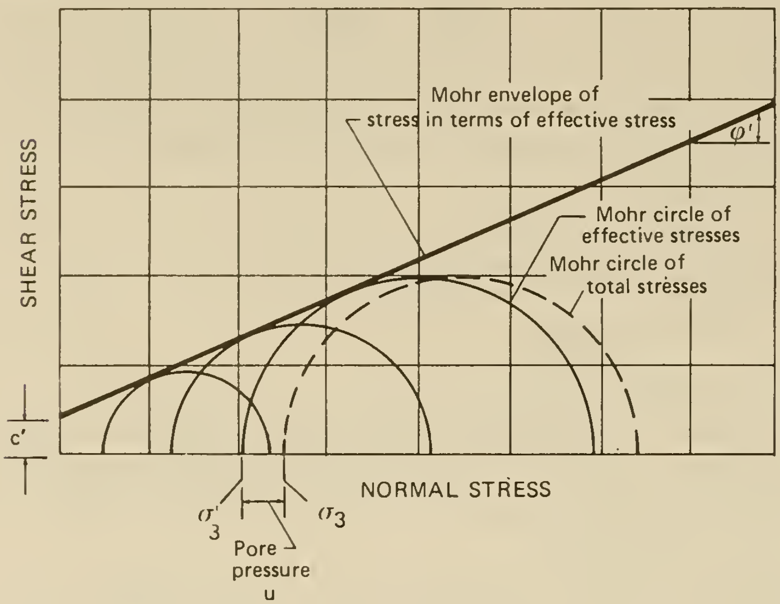
Using the shear strength of a soil for the design of an earth starter dam for tailings disposal is common practice. In this case, the soil can be mechanically compacted to a given density where the shear angle, cohesion, permeability, etc., needed for design can all be determined by laboratory testing. To determine the same physical properties for the tailings is a bit more difficult because it is generally deposited hydraulically with no compaction. If an old tailings pond is available for undisturbed sampling and testing, these figures can be obtained and used to determine the physical properties and geometry of the tailings dam.

If an old tailings pond is not available, such as in the case of an entirely new property, some assumptions must be made as to the density of the deposited material, from either laboratory tests or information from another property with the same grind and rock types. The screen analysis, mineralogy,

pulp density of deposition, and cyclones, if any, affect the material characteristics that determine the shear angle, cohesion, drainage, etc. Soil shear strength is also affected by many test factors including such items as rate and method of loading, principal stress ratios, degree of saturation, drainage, rate of specimen strain, and total specimen strain. In selecting the shear strength parameters that are to be used for specific analyses, an estimate must be made of the probable strains and rates of pore pressure dissipation under field conditions, and a decision must be made whether to use "peak" or "residual" shear strength values to determine the angle of internal friction (ϕ). Generally, if the void ratio (e) is small, peak ϕ is used, and if e is high, residual ϕ is used. More testing is required to determine the shear strength characteristics of soft soil than firm soil.

Triaxial shear testing is a very exacting process that requires good equipment and much training and experience, and it is best left to specialists in this field. The sample must be taken with care in the field, prepared for transport, and transported with minimum disturbance. In the laboratory the sample must be extracted and placed into the machine with extreme care. The unconsolidated, undrained (uu) test is described in ASTM D2850-70 (12) This is an important test because of its use in stability analysis.

Data obtained from shear strength tests are normally presented in terms of effective stresses. During triaxial testing, both total stresses and pore water pressures are measured, the effective stress is the total stress minus the pore water pressure. Plots of effective stress at three different confining pressures for soil specimens at failure are presented in the form of Mohr circles in figure 7. The apparent cohesion (c) is the intercept of the effective angle of the internal friction angle (ϕ) with the shear stress line. For a noncohesive soil this intercept would pass through the origin.




$c'$ = effective cohesion

$\varphi'$ = effective angle of shearing resistance

$\sigma_3$ = total normal stress

u = pore pressure

$\sigma_3'$ = effective normal stress

Shear strength = $S = c' + (\sigma - u)\tan\varphi'$

(Mohr diagram showing envelope of soil strength in terms of effective stresses, and relationship between effective and total stresses.)

FIGURE 7. - Mohr diagram.

## Permeability

Permeability depends on a number of factors. The main ones are--

1. The size of the soil grains.

2. The properties of the pore fluid.

3. The void ratio of the soil.

4. The shape and arrangement of the pores.

Permeability tests can be run on either disturbed or undisturbed representative samples, depending on the use to be made of the samples. If the material is to be moved mechanically and used to build the starter dam with borrow material or raise the dike with tailings sand, the permeability should be measured at the density at which it is to be placed. The foundation material and the beach area of the tailings pond must be measured in an undisturbed condition, because the layering and the difference in horizontal and vertical permeability cannot be duplicated in the laboratory. Granular soils should be measured by the constant head method, and the less permeable material by the falling head method. Range of permeabilities for various soil types are shown in figure 8 in centimeters per second (log scale).

## Consolidation Tests

Consolidation tests determine the compressibility of the soil and the rate at which it will consolidate when loaded. The two most important soil properties furnished by a consolidation test are (1) the compression index ($C_c$), which indicates the compressibility of the specimen and (2) the coefficient of consolidation ($C_v$), which indicates the rate of compression under a load increment. The data from a laboratory consolidation test make it possible to plot a stress-volume strain curve, which often gives useful information about the pressure history of the soil.

To predict the settlement of a structure such as a tailings dam in the field, a method of extrapolating laboratory test results for the settlement analysis is needed. Such analysis requires a specialist, and no further discussion will be included in this text. The reader is referred to Lambe and Whitman (34) and various other texts for comprehensive explanations.

Highly compressible soils should be avoided for tailings areas because of the subsidence of the dam itself and the difficulty of maintaining decant lines and towers where settlement occurs under load. As an illustration of what can be expected of compressible clays, in one case the observed settlements ranged from 100 to 200 percent of the computed settlements.

Consolidation test procedures are detailed in ASTM D2435-70T (11).

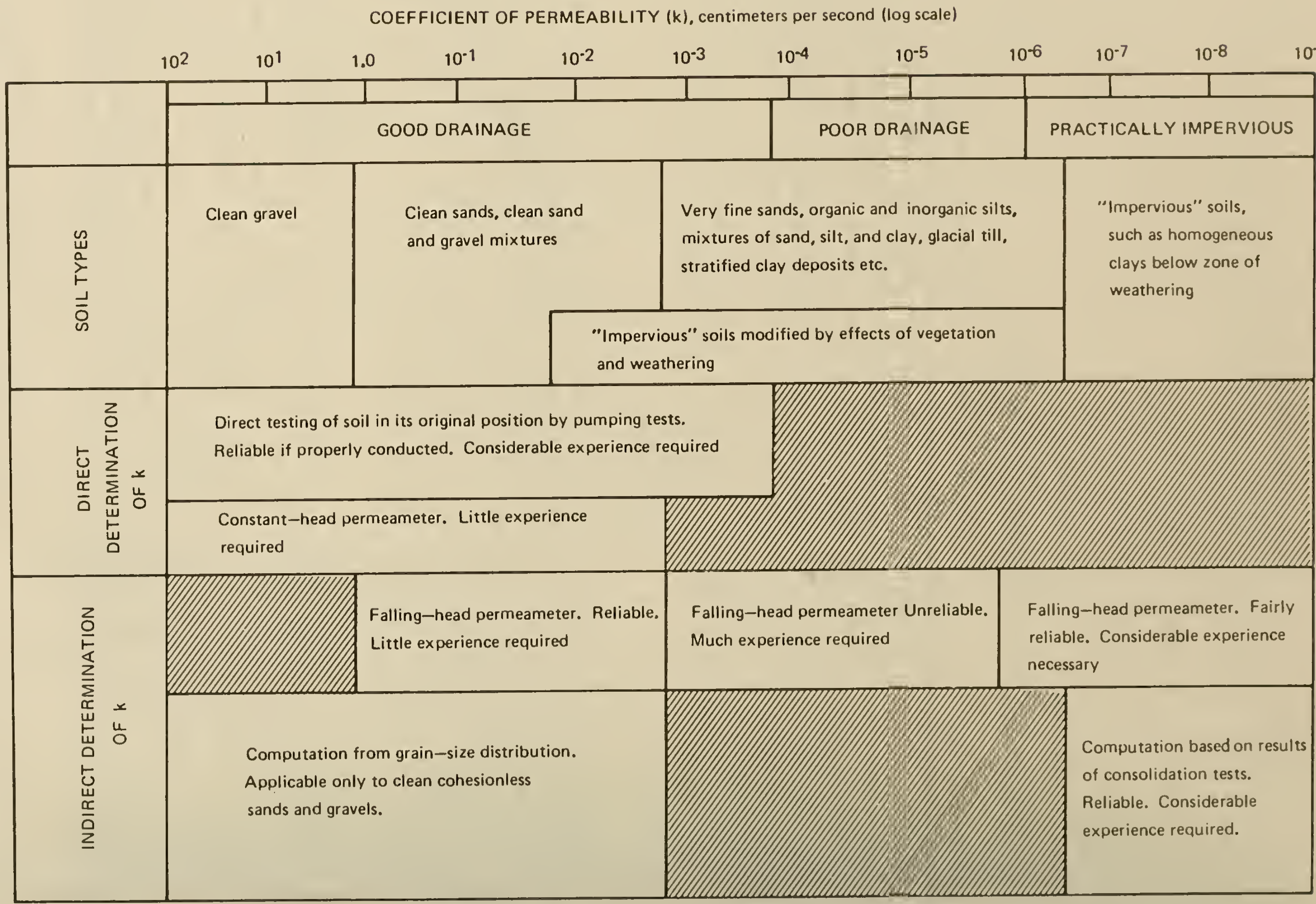



FIGURE 8. - Permeability coefficients for soils.

## Field Testing

### Standard Penetration

The strengths of cohesionless soils may be estimated in the field from the standard penetration test. This test measured the resistance of driving a split-spoon sampler into the soil with a 140-pound weight dropping 30 inches. This sampler is 1.375 inches in ID, 2 inches in OD, and 2 feet long. The number of blows required to drive the sampler spoon 12 inches is called the standard penetration resistance (N value) of the soil. Empirical relationships have been developed to allow correlation of penetration resistance to relative density. This relationship is illustrated in tables 3 and 4. An approximate relationship between the standard penetration blow count (N) and the consistency of the clay is also shown in tables 3 and 4, but should be used only as a guide.

TABLE 3. - Approximate relationship between standard penetration and relative density and consistency for cohesionless soils

| Number of blows (N) per foot of penetration | Relative density |
|---|---|
| 0- 4 | Very loose. |
| 4-10 | Loose. |
| 10-30 | Medium. |
| 30-50 | Dense. |
| Over 50 | Very dense. |

TABLE 4. - Approximate relationship between standard penetration and relative density and consistency for cohesive soils

| Consistency | Very soft | Soft | Firm | Stiff | Very stiff | Hard |
|---|---|---|---|---|---|---|
| Number of blows (N) per foot of penetration | 2 | 2-4 | 4-8 | 8-15 | 15-30 | 30 |
| Unconfined compressive strength (tons/ft$^2$) | Less than 0.25 | 0.25 to 0.50 | 0.5 to 1.0 | 1.0 to 2.0 | 2.0 to 4.0 | More than 4.0 |

### Density

Shelby tubes can be used to obtain the inplace unit weight of cohesive soils in a drill hole in most tailings, sand, and silt containing no gravel or stone fragments. In pits and excavations the sand cone method, ASTM D1556-64 (8), the rubber balloon method, ASTM D2167-66 (10), or the nuclear probe method, ASTM D2922-71 (13), can be used at shallow depths. A nuclear moisture probe is also available for use in field testing.

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## Shear Strength

A vane shear auger is used for measuring the shear strength of a medium-to-soft clay in situ. It consists of four rectangular rigid steel plates arranged at right angles to each other and attached to the bottom of a steel rod. The auger is forced vertically into the soil and is then rotated horizontally. The torque required to turn the auger a sufficient amount to shear the soil on a cylindrical surface at the outer edge of the vanes is a measure of the shearing strength of the soil. Static penetration tests such as the Dutch cone are used in boreholes to evaluate in situ shear strengths. Correlating the information from vane shear and static penetration tests to get the in situ shear strengths is the difficult part of the process and is far from an exact science.

A large-scale direct shear test can be conducted in the field by enclosing a specimen sample of soil or rock within a split box, and applying a vertical load, after which a horizontal load is applied to the top half of the box. The ratio of the shear stress to the normal stress is obtained and analyzed as in the laboratory test.

## Permeability

Measuring the permeability of the soil in situ is a very difficult and expensive operation, requiring considerable expertise. The infiltration test measures the rate of seepage into a drill hole, well, or test pit under a fixed or variable head. The area tested must be below the ground water line or the results will be erroneous.

In the pumping-out test, water is pumped from a well or test pit at a constant rate, and the drawdown of the water table is observed in wells placed at various distances from the pump. This method is quite expensive because of the number of wells required and the pump installation and pumping cost. From these measurements, the soil permeabilities can be calculated. This test is much more accurate than calculations from undisturbed samples in the laboratory because of the large areas that can be measured and because it measures the effects of the natural structures, stratification, and orientation of the grains of the soil in its natural state.

## DESIGN CRITERIA

The procedure for the design of tailings dams will usually involve the following:

1. Select the most promising site in relation to the plant site for low capital and operating cost, ease of operation, and safety. The site must conform to the general topography of the area, land available, and habitation, which will determine which of the following types of dams are required:

   A. Cross valley (all extraneous drainage through the impoundment).

B. Cross valley, two dams (all extraneous drainage bypassed beneath the embankment by pipeline).

C. Flat-area dams on three or four sides.

D. Cross valley, water-type dams.

E. Incised and built up on four sides.

F. Large area with a series of water-type dams between low hills or in saddles, no spigoting.

2. From the topographic map of the storage area, calculate the live storage volume and plot the results in a storage volume-elevation curve. On the elevation side, plot the time also because this is a very important factor. If the given area cannot hold the entire mine output, a decision must be made as to how high the embankment is to be built and its ultimate capacity.

3. Determine drainage upstream from starter dam, and design decant lines and towers to handle total flow expected.

4. In a new property, a sample of tailings from the pilot mill should be checked for gradation, segregation from spigoting or cycloning, inplace density, and horizontal and vertical permeability. The gradation of the material will determine whether the tailings can be used for embankment construction (figs. 9-10), as follows:

A. Coarse (0 to 15 percent minus 200 mesh), good.

B. Medium (15 to 50 percent minus 200 mesh), generally good.

C. Fine (50 to 80 percent minus 200 mesh), questionable--only under special conditions.

D. Slimes (100 percent minus 200 mesh, and 25 percent minus 0.2 micrometer), retained by borrow material only or water-type dam.

5. Select a trial embankment section that utilizes the most economic and readily available construction materials. Check the sufficiency of this selection by--

A. From the laboratory tests obtain the density of the construction material at 95 percent of Standard Proctor, the permeability, and the shear strength and cohesion of the construction and foundation material, and make a stability analysis using the pore water pressures within the embankment and foundation.

B. A flow net should be drawn to estimate the pore water pressure resulting from steady seepage within the embankment and pervious foundation.

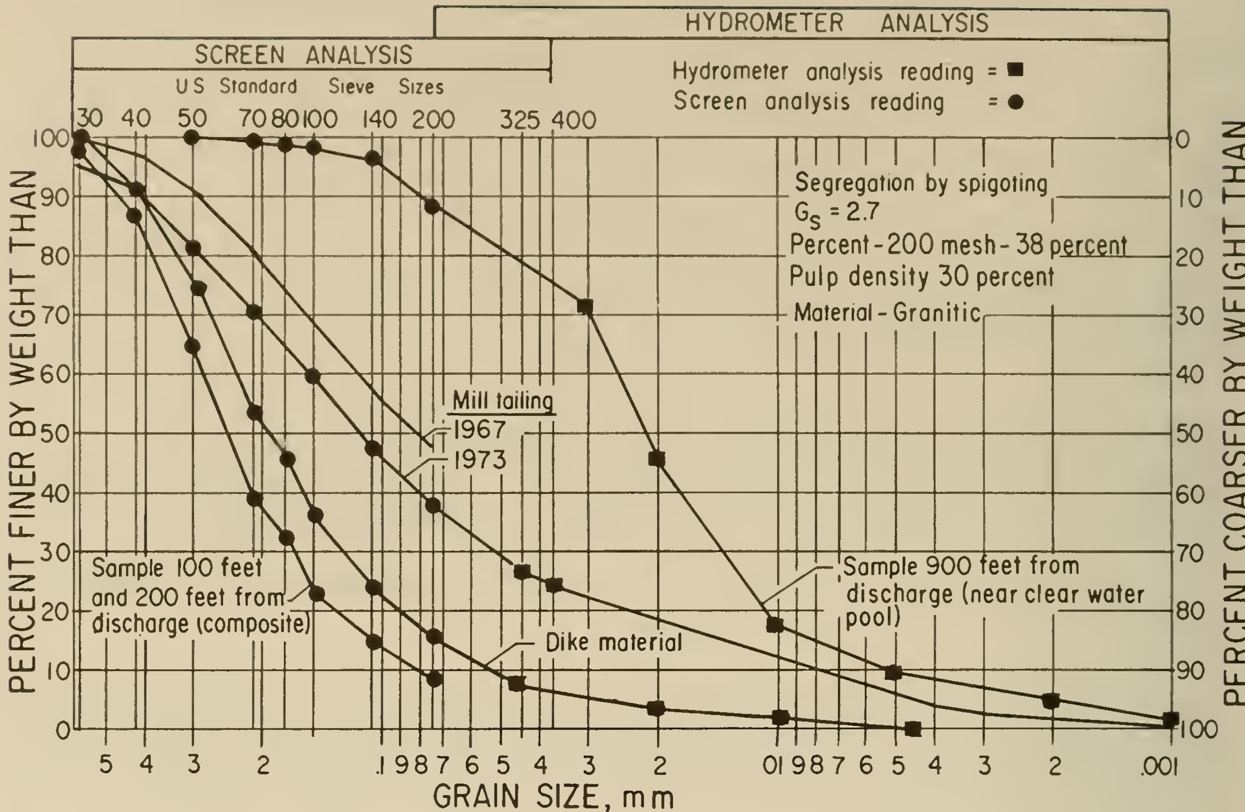


FIGURE 9. - Gradation of metal mine tailings—coarse grind, low pulp density, mine A.

C. If the foundation contains compressible strata, foundation pore pressures estimated on the basis of consolidation-time theory should be taken into account in the analysis and should be checked by field measurements during and after construction. (Properly constructed blanket drains should eliminate all pore water pressure from the foundation.)

D. Repeat the stability analyses until a section has been found that has the required factor of safety.

Again it must be emphasized that the parameters used in the analysis are of paramount importance for accuracy and are the most difficult to obtain for a new property. Probably the most difficult and the most important is the phreatic line, which can be found by making a model of the tailings pond and getting horizontal and vertical permeabilities and using the finite-element method of determining the flow through the embankment (31). In making the model, the material must be discharged at the same grind and pulp density as it will be in actual operation. The permeabilities obtained in the samples of unconsolidated material will not be the same as that of the same material after consolidation, but the relative horizontal to vertical permeability will be approximately the same. The vertical sample could then be loaded in the consolidometer and permeabilities measured at various loadings to simulate

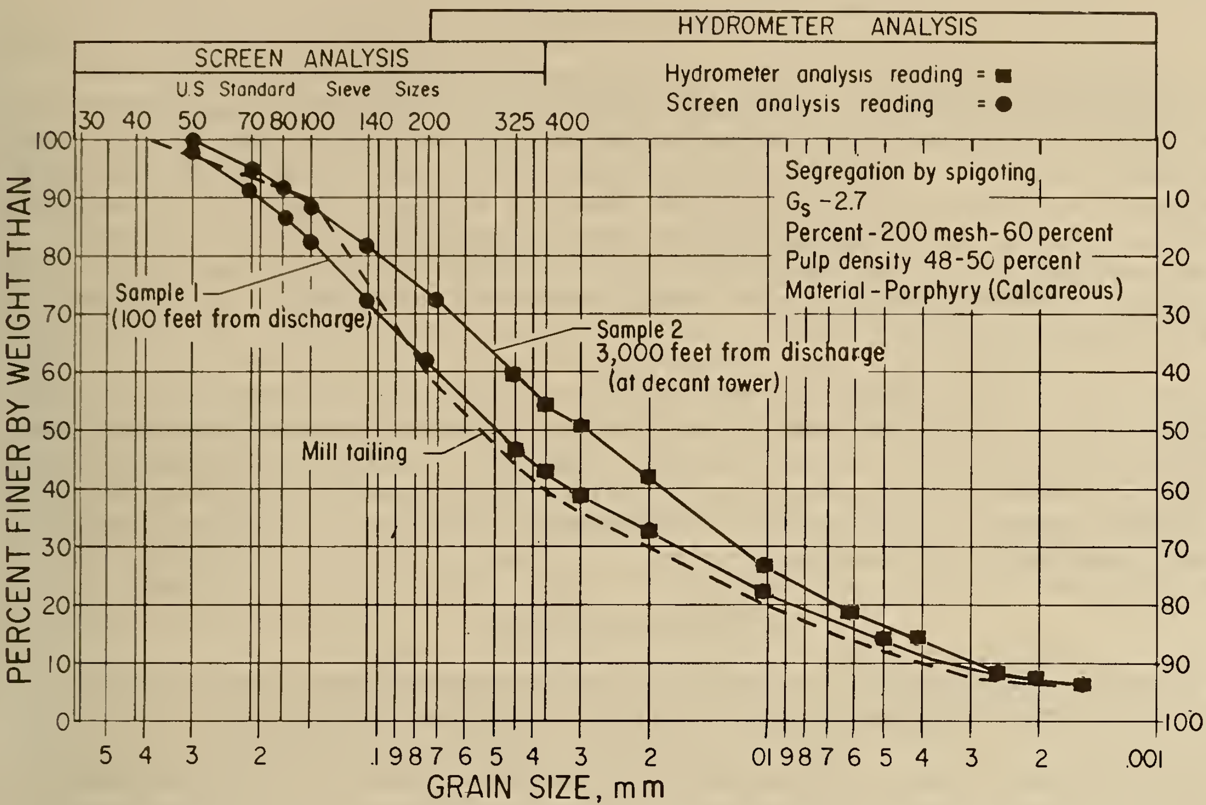


FIGURE 10. - Gradation of metal mine tailings—fine grind, high pulp density, mine B.

various embankment heights, and from this the horizontal permeabilities could be calculated at the same heights.

Small-volume, low-height embankments are quite simple to design and operate. If borrow material is readily available, an all-borrow dike may be desirable, or a small starter dam with spigoting around the periphery and the embankments raised with tailings sand may be the most economical.

Generally when the ultimate height of the dam is to be only 50 to 100 feet, stability is not a problem, provided good operating procedures and reasonable slopes are used and the annual rise is kept low. The site selection sampling and testing are still necessary to be sure that the foundation is sufficiently strong to accommodate the weight to be placed on it.

Embankments need a starter dam to provide sufficient freeboard to prevent overtopping at the start and to provide water storage for clarification and reclaim. Large operations in a narrow valley may require an extra-large starter dam because of the small initial acreage, or they should have two or more sites in use at the start of operations. Annual rise is very critical and can cause trouble if it is too rapid. As much as 10 years' leadtime may be required before a single site could handle the total tonnage in a large operation.

## Upstream Method

Among the more popular types of tailings dam configurations is the upstream method, which gets its name from the fact that it is constructed from the starter dam in lifts which are successively placed on the dam in an upstream direction. With the upstream method the starter dike is at the downstream toe and should be more pervious than the tailings sand or have drains on the upstream side of the starter dam that would prevent the water from accumulating there. With the drains as mentioned, the starter dam can be built of a suitable material with a high shear strength even if it is relatively impervious.

The coarse sand spigoted around the periphery of the tailings pond makes a good material for embankment construction. It should be well compacted and kept below saturation to prevent liquefaction. A stability analysis of the proposed embankment is necessary to determine if it can be safely built. If the design proves to be unstable, additional compaction or a flatter downstream slope may be necessary.

## Downstream Method

The downstream method of building tailings dams results in the centerline of the dam moving downstream as the height of the dam increases.

This method of building tailings embankments should have a relatively impervious starter dam at the upstream toe if water retention is important. Cyclones are used for production of the building material, which drops out on the downstream side, and the overflow, which is piped 100± feet toward the upstream side. One disadvantage of this method is the cost of maintenance and operation of the cyclones. Another disadvantage is the successively larger amounts of material necessary with each foot rise in height of the embankment. The cycloned product may not be very uniform because of cyclone wear and pressure fluctuations to the cyclones. Drainage blankets should be placed downstream from the toe dam on top of the natural soil prior to depositing the cycloned sand, to prevent buildup of water within the structure.

## Centerline Method

The centerline method is similar to the downstream method in that a cyclone is used to separate the sand and fines, but the centerline of the dam remains above the starter dam and is ideally suited to small operations where the pipe can be suspended vertically in the air with cyclones attached directly to it. Another system used to construct the centerline method is to have the cyclone cluster mounted on a truck (similar to that used on some downstream ponds) that moves slowly across the top of the dam depositing the underflow in front of the truck in the direction of travel and the overflow onto the upstream side of the dam. In this way the truck makes its own roadway and can raise the embankment as high as 30 feet in one pass across the dam, provided there is room for the overflow fines. This has the advantage of concentrating all the cyclones in one spot for easier maintenance and

operation and better control of the pressure to the cyclones. It also requires very little daily care.

Some research should be done on these last two methods to determine how high the dams can be built and the location of the phreatic surface in the embankment.

## Upstream Method With Cyclones

Another method that is being used successfully is the upstream method with cyclones. This method is similar to the upstream method with spigots in that it has a starter dam and drains if needed, but the slurry goes to a cyclone mounted on a 12-foot tower upstream from the header pipe. The underflow drops out directly below the cyclone and around the tower, and the overflow is piped 150+ feet into the pond. The sand forms a slope of 3:1. The cyclones are raised three successive times to an embankment height of 35 feet, the top is leveled off to a 30-foot-wide berm and roadway, and the header pipe is then raised and the cycle repeated. The overall slope on the downstream face is about 4:1, which is very flat for a metal mine tailings embankment but very good for stability.

Some research needs to be done on the stability of this type of operation because the effect of the weight of sand on top of the slime is not known. The sand-slime interface is jagged, allowing the water that is squeezed out of the slime to escape easily into the sand. This method also places the clear water pool close to the dam, but the seepage through the dam probably is limited by the low permeability of the slime. This system also has the clear water-soil contact at the upstream edge of the pool, which allows substantial tailings water seepage if the natural soil is pervious. This is a very serious disadvantage where contamination of the ground water must be reduced to the minimum.

Other sections contain details on site and material investigations and methods of settlement, seepage, and stability analysis for tailings embankments. Instrumentation, slopes, and factor of safety under various conditions are also suggested.

## Starter Dam Design

The site investigation, including trenching, drilling, sampling, and laboratory testing, should indicate the type, quantities, and physical properties of the foundation material in the dam area. From this information and the properties of the tailings, the starter dam can be designed. The material available for construction of the dam is most frequently borrow material from within the disposal area. If this is mostly clean sands and gravels with high permeability, a pervious starter dam can be built. If it is predominantly clay mixed with silts, sand, and gravels, an impervious starter dam with filters and drains should be built. Overburden or waste rock from open pit or underground operations can also be used. The materials used should be those that can produce adequate stability at least cost.

## Pervious Starter Dam

Excavation for the base of the starter dam should be down to a competent soil that will withstand the weight contemplated. All the organic soil, trees, and brush should be removed. On a smooth rock foundation with a 5- to 10-percent slope, a trench cut into bedrock may be needed to key the dam to the rock. Foundation defects such as open cracks in the bedrock, clay seams, buried coarse talus deposits, or pervious foundation soils should all be remedied. Loose and pipable material should be excavated, and open cracks should be filled to prevent piping under the dam.

All the possible problems and conditions for all situations cannot be contemplated. Actual treatment of the foundation depends on conditions exposed in the field and must be solved there. Seepage through or beneath the starter dam in this case is not bad except that it must be controlled so that it does not lead to piping. On deep alluvium most of the seepage would go out the bottom of the pond with part of it flowing under the dam.

A pervious starter dam should have a permeability of $10^{-2}$ to $10^{-3}$ centimeters per second, but the main criterion is that it have a higher permeability than the sands it is retaining. It is necessary that the starter dam <u>not</u> retain water so that the phreatic surface hits as low as possible on the upstream face and does not emerge on the downstream face. All the water that reaches the starter dam must go freely through it to a collection pond below the downstream toe. The sand-gravel mix must be placed in thin layers and compacted to 95 percent of Proctor to insure stability while allowing flow through the dam. The borrow pits in the material used for construction of the dam should be tested for permeability in the laboratory at Standard Proctor density and the material should be placed in the dam so that the permeability increases downstream and the overall permeability is greater than that of the sand being impounded.

## Impervious Starter Dam

If all or most of the borrow available for construction within economical hauling distance of the site is a relatively impervious material, or if the "downstream method" of placing tailings is to be used, an impervious starter dam should be built.

The method of construction for the impervious starter dam is the same as for the pervious dam. Compaction should be 95 percent of Standard Proctor, and the foundation excavation and preparation should be the same. For the ordinary <u>upstream</u> method of placing sands, the starter dam should have drains to catch all the seepage water and let it pass freely under the starter dam in pipe or blanket drains. Under <u>no</u> conditions should the starter dam retain water against its upstream face because it would become saturated and unstable. Under these conditions the seepage could emerge high on the sand face above the top of the starter dam, and remedial measures would be necessary. These remedial measures are described elsewhere but are no substitute for proper drainage, design, and construction. The ultimate height that the dam could be built is materially reduced if a high phreatic line is generated.

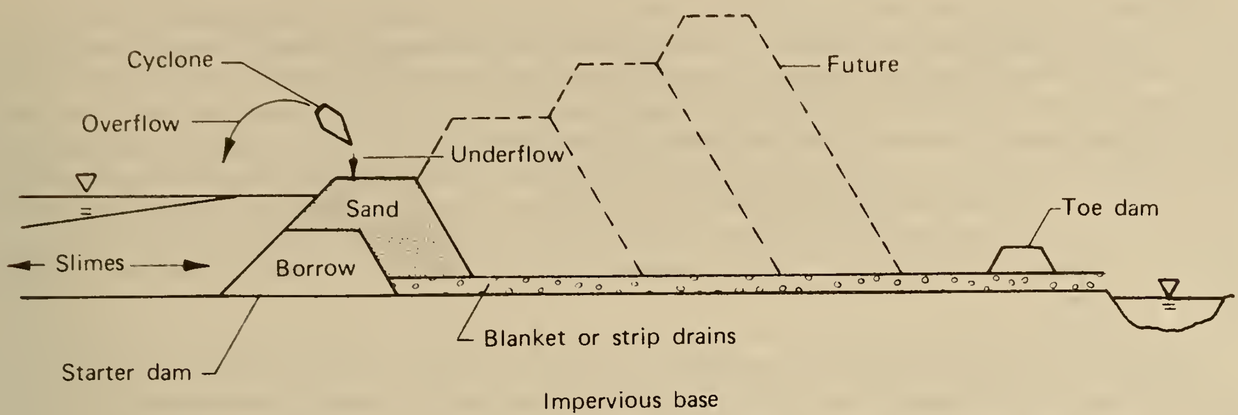


FIGURE 11. - Downstream method showing both dams.

With the downstream method, the starter dam is at the upstream toe of the completed dam. It can and should be impervious relative to the sand and retain water as much as possible. The seepage that eventually goes through and over the top of the starter dam will move down through the more pervious sand and into the drains between the starter and downstream toe dam (fig. 11). The stability of this starter dam is not a problem because it eventually is completely surrounded by tailings sand on its top and downstream and by slimes upstream.

The area between the upstream starter dam and the downstream toe dam must have blanket or strip drains to catch all the seepage and drain it out to a holding pond where it can be recycled or discharged. These drains would not be necessary if the cyclone sand were >100 times the permeability of the starter dam.

The steeper the terrain and narrower the valley, the higher the starter dam must be. If many years of leadtime are available and the area can be raised slowly, a small starter dam can be used, allowing the sand to build up to an elevation where the area is large enough to take the entire output continuously.

Two complete and separate areas are desirable in order to allow deactivation of one area to drain, build sand dams, raise pipes, etc., while the other area is activated. At startup, with large tonnage, the fill time is quite short even with two areas, and drainage may be the limiting factor. They may not drain fast enough to allow the necessary time to raise the dams with the sand. When the original ground is steep (5- to 10-percent slope), the drains may not be able to handle the water because the pond area is small and the rate of rise is fast. In this case, standby areas should be provided to take care of emergencies.

## Drainage

From the designer's viewpoint, it is desirable to promote drainage of water from the tailings pond in order to keep the phreatic surface as low as

possible and help the consolidation and stability of the embankment. For this reason, the relatively pervious tailings dam is the most common design used. It is also the cheapest because it can be built from the coarse fraction of tailings or from readily available borrow material, such as is now used in the Florida phosphate mines. The impervious tailings dam is the least common type and is only used where it is necessary to retain polluted water or low-density solids that are slow to consolidate. In either type of dam, the stability of the dam is of paramount importance and necessary provisions must be made to control seepage through and under the embankment and to control surface runoff into the pond.

Water control in a tailings pond is probably the most critical single design item affecting slope stability, followed by (1) weak foundations, (2) slopes that are too steep and too high, and (3) low density of the material in the dike and on the beach. Failures are often caused by overtopping of pond water, by piping of fine materials because of seepage through the embankment or foundation, and by piping along the outside of decants and culverts because of the lack of or inadequate seep rings. The physical rupture of a decant line or decant tower allows piping into the line from the tailings, which is nearly impossible to remedy. Insufficient leadtime in constructing a disposal area can cause too rapid annual rise of the pond and bring about sudden failure. Poor operating practices, such as failure to control the size or location of the clear water pond, can do the same thing.

Unwanted seepage through the bottom of a tailings pond in a relatively level area with deep pervious alluvium can be tremendous at the clear water-soil contact. A layer of slimes reduces this seepage considerably, but with a normal spigoting operation the slime is below the area of contact, leaving a water-soil contact 50 to 100 feet or more wide unless special effort is made to place a slime layer over the entire area first.

## Drains

The position of the phreatic surface in an embankment is very critical for its stability. Of all the factors controlling stability, the control of water within the embankment is by far the most important. For example, a slope composed of cohesionless sand having a total unit weight of 120 pounds per cubic foot and an angle of internal friction of 35° will remain stable at an angle of 35° regardless of the height of the slope, providing the phreatic surface is below the toe of the slope. If the phreatic surface is raised to exit the downstream face of the slope, the steepest stable angle is reduced to approximately 18° (20).

A relatively impervious starter dam very definitely requires drainage on the upstream side, and a pervious starter dam does not necessarily preclude the need for drains. Should the pervious starter dam become ineffective and there is no drain to fall back on, the phreatic surface can rise and seriously reduce stability.

Two of the basic types of drains are (1) the blanket drain, which is coarse gravel sandwiched between protective filter material which carries all

the water through and beneath the dam, and (2) the perforated pipe drain, which is surrounded by gravel and a protective filter which collects the drainage and carries it through the dam. The choice of which one to use depends on the availability of drainage material, drainage capacity required, foundation conditions, and construction cost. Variations and combinations of these two methods can be used, but they must be carefully constructed, and the material used for filters must have proper grain-size graduation.

## Blanket Drain

This type of drain would be used in a cross-valley dam with either a pervious or impervious starter dam, where bedrock or a relatively impervious base is close below natural ground level and where the upstream method of dam building is to be used utilizing the tailing sands. The purpose of this blanket drain is to intercept the water that moves downward out of the tailings as well as any springs or artesian water that may come up from below. If springs are found in site investigation or artesian water in drill holes, either from the rock or below a stratum of impervious clay, the blanket drains should have capacity to remove all this water plus additional capacity for that which may not have been discovered. It is very important that this water be removed because in mountainous areas it could have a high head and if trapped below the slime layer in a tailings pond it could exert tremendous upward pressure and greatly reduce the factor of safety of the embankment.

The drain consists of a layer of clean gravel up to 18 inches thick extending from above the upstream toe to below the downstream toe and wide enough to cover the main valley bottom. This gravel drain is protected by a 9- to 12-inch filter layer of clean sand and gravel both above and below. An additional drain of unprocessed sand and gravel up to 3 feet deep is also placed upstream to extend the drain area as far as deemed necessary to catch all the seepage (fig. 12).

These drains must have a catchment ditch filled with cobbles to intercept the drainage and prevent erosion on the downstream face. Where the downstream slope of an embankment is composed of fine-grained materials, water should not be allowed to flow out of this slope. Lowering the phreatic surface increases the stability, permitting the use of steeper slopes, and reduces the volume of construction material needed. In cold climates it is especially important that the drain water be directed through a drainage blanket below the compacted soil so that it will not freeze and raise the phreatic surface causing the entire embankment to become saturated behind a frozen blanket of soil on the downstream face of the starter dam.

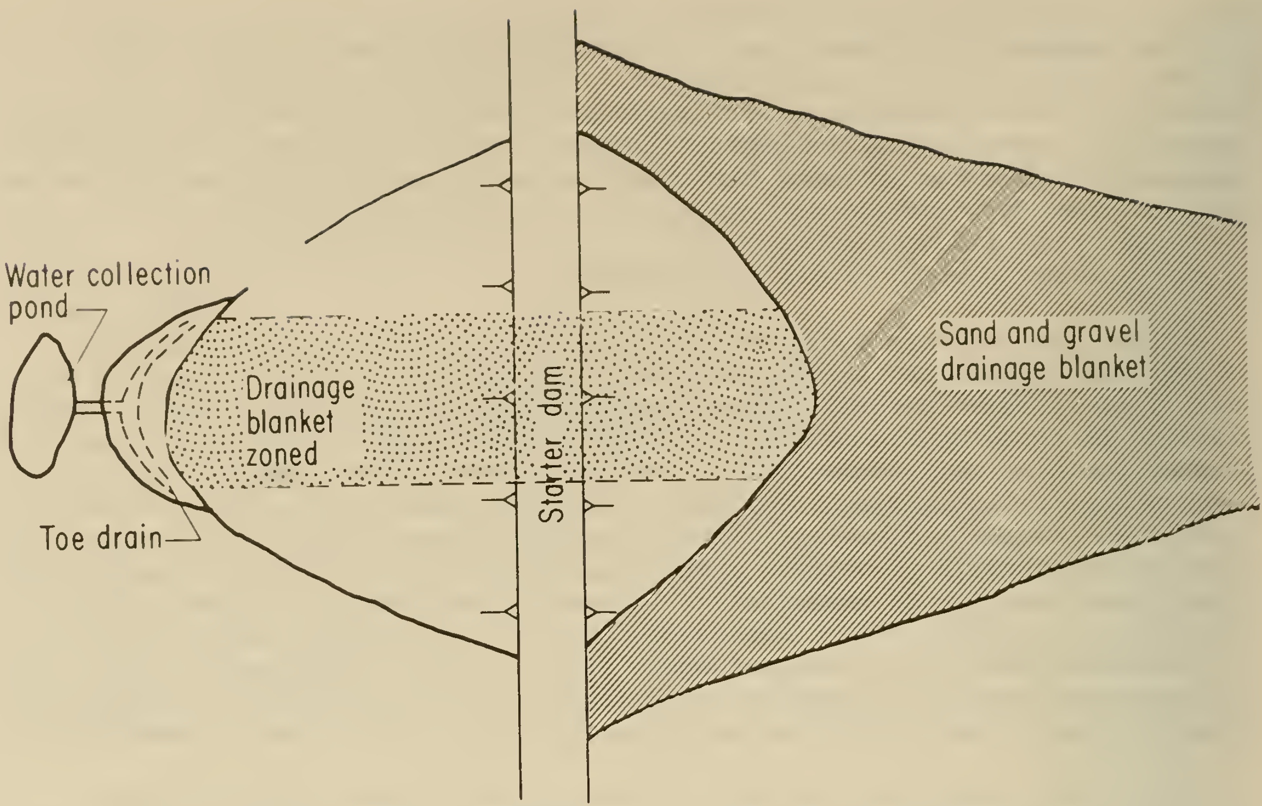


FIGURE 12. - Blanket drain.

## Pipe Drain

Where drainage pipes are to be used, the pipes should be designed to withstand the maximum anticipated load of the overlying tailings. When perforated pipe is used, it should be perforated on the bottom half only and laid with the perforations down, with a bed of gravel both top and bottom and graded filter surrounding the gravel (fig. 13). The diameter of the perforations should not be larger than one-half of the 85-percent size of the drainage material surrounding the pipe. Pipe drains can be very satisfactory with a good foundation and careful construction, but the blanket or strip drains may be more fail-safe. Various arrangements of pipe drains can be made. A perforated pipe parallel to the upstream toe of the starter dam with one or more solid pipes through the dam to the downstream toe is the simplest. This same arrangement can be used as a collection for drains up to 600 feet long running parallel to the valley at right angles to the dam axis and spaced at 50- to 100-foot intervals along the valley floor and walls (fig. 14). Pipes through the starter dam should not be perforated and should have at least three cutoff collars that extend at least 2 feet from the pipe to prevent "piping."

If the foundation beneath a tailings embankment is compressible and differential settlement is possible, pipe drains should be avoided. The stresses may result in opening pipe joints or breaking the pipe, which might allow internal erosion.

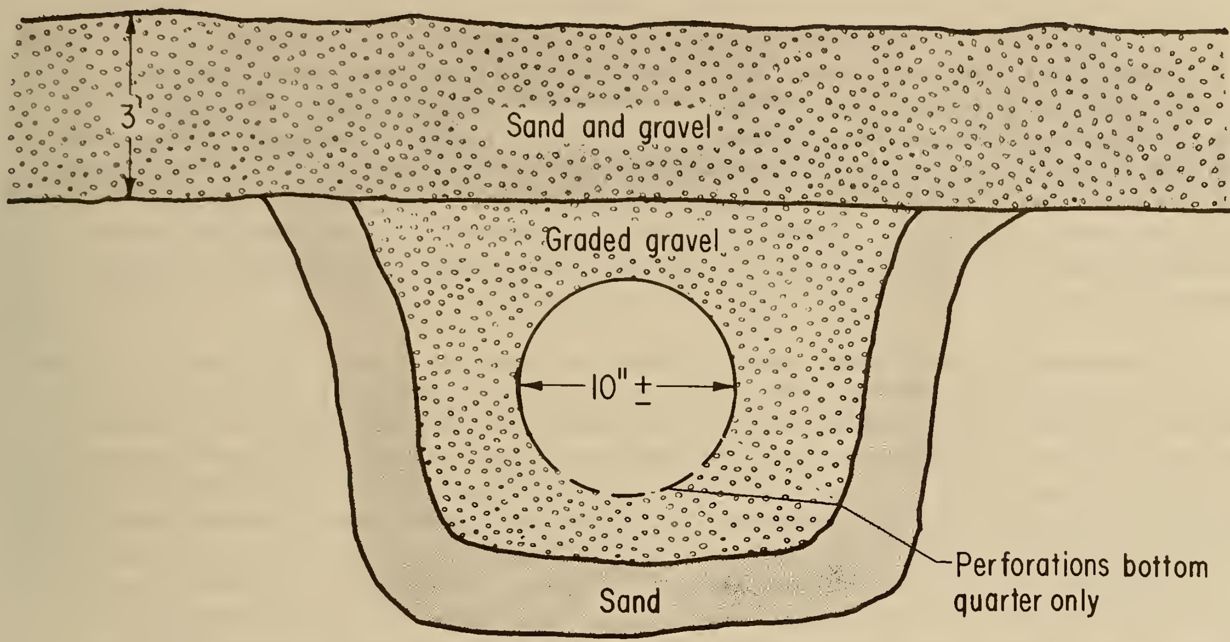


FIGURE 13. - Pipe drain.

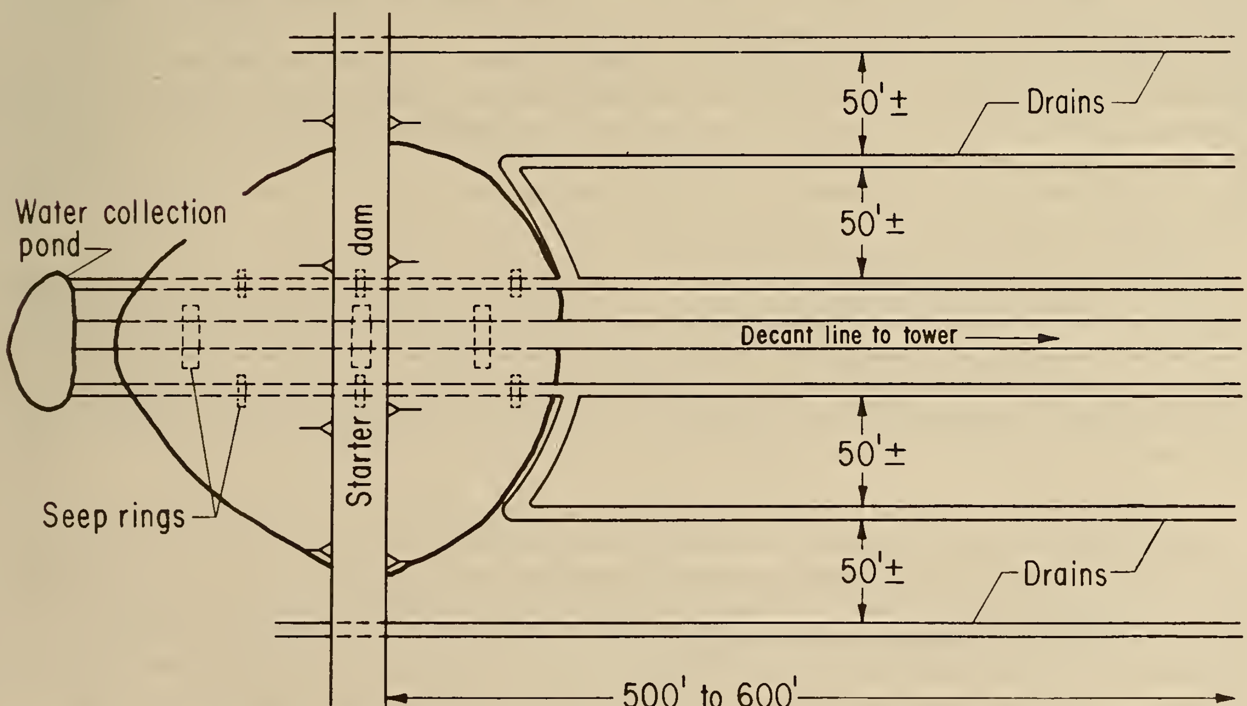


FIGURE 14. - Pipe drain layout.

## Strip Drain

Strip drains are the same as blanket drains in design and construction except that they are narrow strips of drain material laid in the foundation prior to dam construction. They are laid out to carry drainage through the dam and to outlets beyond the downstream toe of the embankments. The drains are laid out in strategic locations to catch the drainage and must be arranged according to the contours of the foundation. Strip drains can be used upstream from the starter dam in the same manner as blanket or pipe drains.

If blanket or pipe drains are placed as much as 500 to 600 feet upstream from the toe of the starter dam and spigoting is started with tailings, the slimes begin settling on the top of the drains in the first 100 feet from the upstream toe of the starter dam. As the sand builds up, this fine material continues to move uphill in an increasingly thicker layer. The layer of fines or slimes is quite impervious even in thin layers, and under consolidation and drainage (which would be very good over the drain) it can have a permeability as low as $10^{-6}$ or $10^{-7}$ centimeters per second. This layer of slimes over a drain renders the drain useless except for the first few feet upstream of the starter dam. For this reason when drains are constructed upstream any distance above the starter dam, the tailings should be cycloned and the coarse underflow should be repulped and spigoted on the dam to make a beach of very pervious sand that will cover the drain. Since this coarse sand will have a slope angle of 4+ percent, depending on grind, etc., it will require a lot of material and come up quite high on the starter dam. After the drain is well covered, spigoting of unclassified tailings can be started; the blanket of slime will not cover the drain, keeping it free and operating for the life of the dam. This type of blanket drain would be used only where bedrock is relatively shallow and the natural soil is saturated or will become saturated from the tailings pond.

In areas where the bedrock is 100 to 500+ feet deep and the soil is very pervious ($10^{-2}$ to $10^{-4}$ centimeters per second) the blanket, strip, or pipe drains extending upstream from the starter dam would not be used because the seepage through the bottom would go down toward bedrock and not follow the drain. Nearly every mine has a different set of conditions, and each tailings area must be designed accordingly.

Because of the layering in a spigoted embankment, the permeability in the horizontal direction may be as much as 5 to 10 times that in a vertical direction, especially if the grind is coarse and the pulp density is low. To determine the seepage from the pool and from the spigoting on the beach, a flow net should be used to estimate the seepage rate to the drains. The quantity of seepage will depend on the permeability values, hydraulic gradient, and area of flow. In some embankments and possibly all of them, the piezometric head from the downstream toe up and under the beach (on a large dam 500 to 600+ feet distance) is determined more by the water flowing on the beach during discharge than by the water escaping from the pond area. The water in the pond is contained in a saucer of slime with the permeability lowest at the center of the pond and increasing toward the beach. Because of this, control

of the height and location of the water pool is important in controlling the seepage. An increase of 1 foot in the pond will flood 200 to 600 feet of beach, which has a higher permeability than that area at a lower elevation. To insure that the drain design is adequate, the upper range of the coefficients of permeability of the embankment should be used in these calculations. In the same way the lower range of permeability of the drainage blanket should be used to be sure that the blanket has a greater capacity than the calculated seepage of the embankment. In these calculations the permeability of the foundations must be taken into consideration also. With a high water table, water can enter the drainage system from the foundation strata, thus increasing the required capacity of the drainage system. Layers of impervious materials in the foundation strata will restrict seepage into the foundation.

Calculating the thickness and width of blanket and strip drains is probably worth the effort from a cost standpoint because the difference of cost between 1 foot and 2 feet of gravel over a large area could be considerable. The drains should be as large as practicable considering the cost and availability of materials. They should be uniform and continuous and constructed of the proper gradation of materials, without which they could become useless.

Granular materials incorporated in underdrainage systems should be compatible with the properties of the seepage water they are designed to carry. Drainage materials composed of carbonate rocks are unsuitable if the seepage collected by the system is acidic.

Blanket drains and strip drains should be designed to be capable of passing full design flow when the phreatic surface within the drain is at or below the upper surface of the drainage material.

## Filters and Transition Zones

A filter or "protective filter" is any porous material that has openings small enough to prevent movement of soil into the drain and yet is much more pervious than the soil it is protecting. Transition zones are also filters between a very fine and a very coarse material where several different filter materials have to be used. In the construction of a zoned embankment with the permeability increasing in the downstream direction, filters and transition zones should be placed between layers of significantly different gradation to prevent piping and subsurface erosion (fig. 15).

Each situation is different, and the filter design should be governed by field conditions, particularly by the gradation of the soil to be protected and the material protecting it. Two different gradations of filters would probably be used where coarse mine rock was to be placed on the downstream toe of a tailings dam to prevent seepage. Figure 15 shows idealized sections of filters.

Experiments have shown that filters need not screen out all the particles of the soil but only the coarsest 15 percent, or the $D_{85}$, of the soil. These coarser particles ($D_{85}$ and larger) will collect over the filter openings and screen out the smaller particles. Therefore, the screen or holes in a

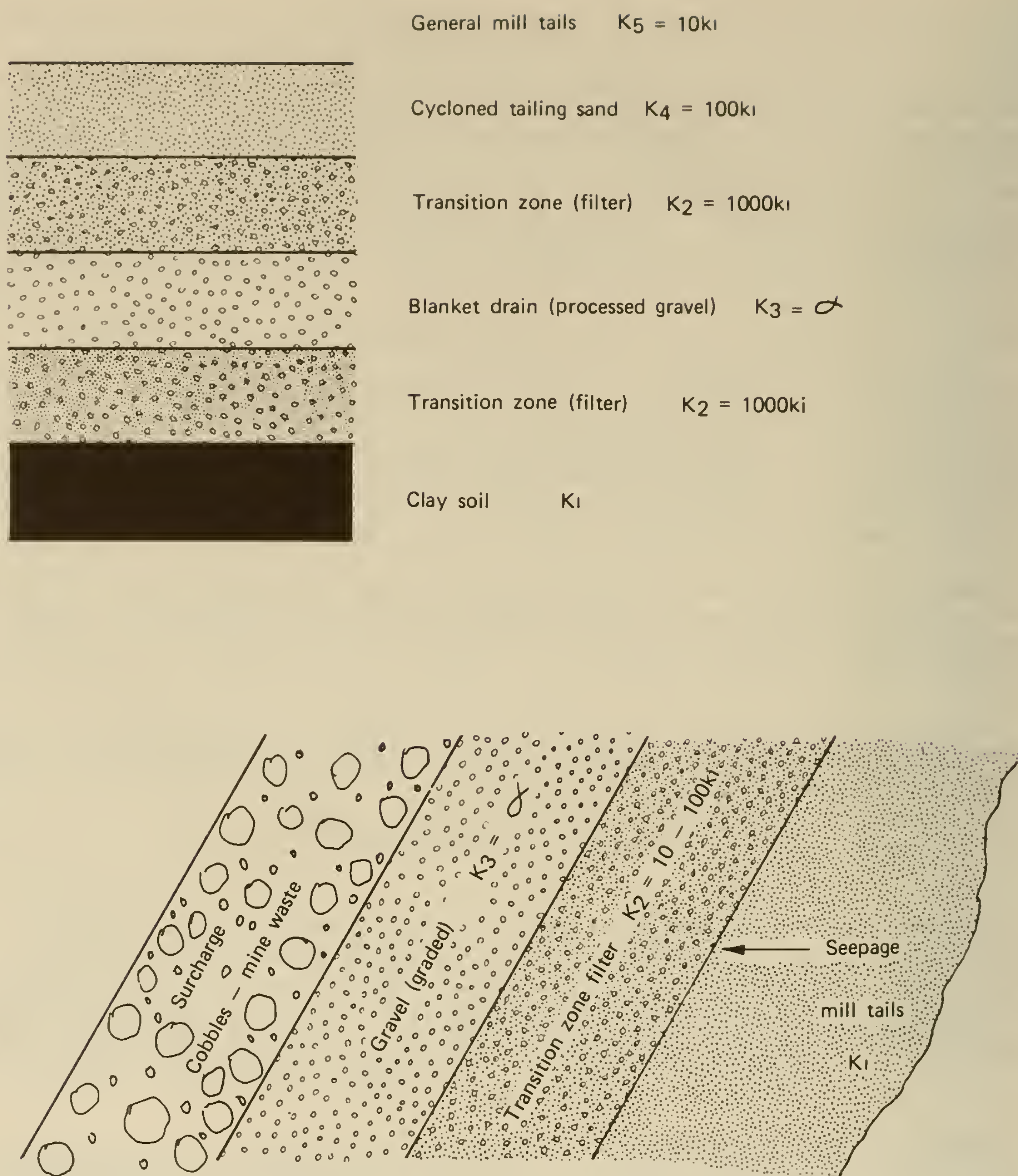


FIGURE 15. - Transitions and filters protecting drains.

perforated pipe must be smaller than the $D_{85}$ size of the soil. By the same reasoning, if soils are used as filters, the effective diameters of the soil voids must be less than $D_{85}$ of the soil being filtered. Since effective pore diameter is about 1/5 $D_{15}$, then $D_{15}$ filter $\leq$ $D_{85}$ soil. The filter must provide free drainage, and since the permeability coefficient varies as the

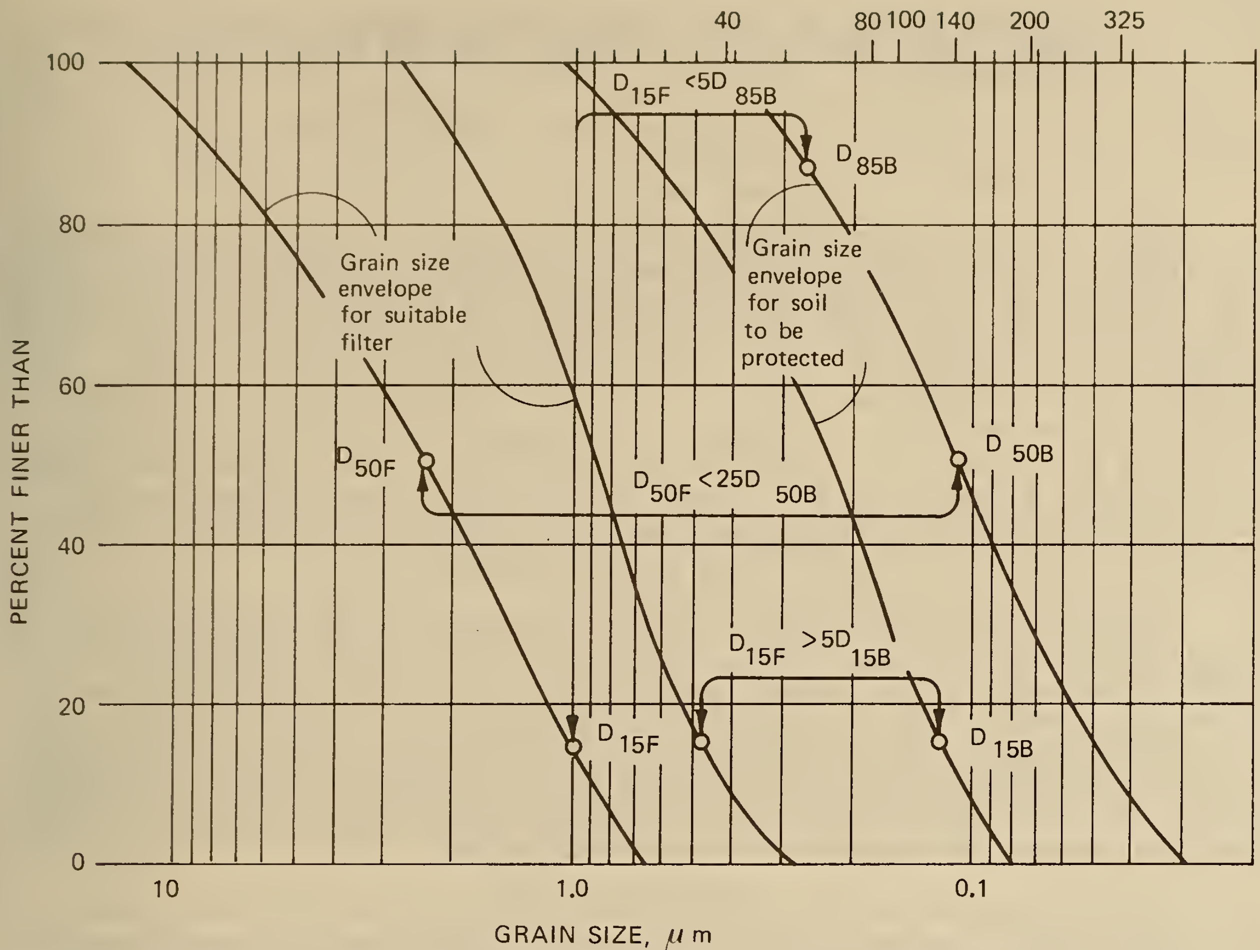


FIGURE 16. - Screen analysis of four filter materials.

square of the grain size, the ratio of permeabilities of over 20 to 1 can be secured by $D_{15}$ filter $\geq 5D_{15}$ soil.

To satisfy the criteria for filter materials, the following rules should be applied as illustrated in figure 16.

RULE 1: $\frac{\text{The 15-percent size of the filter}}{\text{The 85-percent size of the protected soil}}$ should be less than 5.

RULE 2: $\frac{\text{The 50-percent size of the filter}}{\text{The 50-percent size of the protected soil}}$ should be less than 25.

RULE 3: The filter material should be smoothly graded; gap-graded materials should be avoided.

RULE 4: $\frac{\text{The 15-percent size of the filter}}{\text{The 15-percent size of the protected soil}}$ should be greater than 5.

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RULE 5: The filter should not contain more than 5 percent of particles, by weight, finer than the No. 200 sieve, and the fines should be cohesionless.

RULE 6: The coefficient of uniformity of the filter should be equal to or less than 20.

Rules 1, 2, and 3 assure that the protected soil does not pass through the filter. All filters allow small amounts of fines to pass and collect in the outlet pipes, so provisions should be made for flushing out the pipe if at all possible. Rules 4 and 5 are to assure that the permeability of the filter material is high enough to take all the seepage from the protected soil. Attempts have been made to develop a universal filter that will filter the finest soil and yet have a $D_{85}$ large enough that it will not pass through the 5/16-inch perforations of commercial drainage pipes. These filters have such a wide range of size--that is, a very flat gradation line and high $C_u$--that the particles segregate during handling and placement. A filter with a $C_u$ of 20 or less is not as susceptible to segregation (rule 6).

Transition zones and filters are very important in the design and construction of some tailings ponds, particularly where drains are required either upstream from and through the starter dam or in the starter dam construction itself. They are especially important where a water-type dam is to be constructed, and drains are built into the dam itself. Where remedial measures are necessary for seepage emerging on the downstream face of a tailings dam, a protective surcharge of any available material from sand to coarse mine rock can be used with protective filters and drains.

The thickness of filters required varies with the head of water from a few inches with a low head up to 10 feet with a high head in a water-type dam. The filter thickness also varies depending on whether it is placed with construction equipment or by hand. Steeply inclined filters placed with construction equipment should be wide enough to accommodate this equipment easily, while on level ground these filters should be at least 3 feet thick. Filters placed by hand should be at least 6 inches thick.

In summary, filters must be of proper gradation to protect the soil, have a permeability greater than the soil being protected, and have an increasing permeability in the direction of flow.

## Relief Wells

Under certain conditions, relief wells are the logical method to reduce the pore water pressures beneath an embankment. One such situation is an embankment on a relatively impervious base overlying a pervious stratum fed by either artesian or pond water. If the pervious stratum gets plugged downstream, the head could become high enough to exert considerable uplift on the embankment. Relief wells within the embankment could be drilled, and the water pumped out into ditches or piped downstream. These wells function better if they are in the embankment rather than at the toe. Relief wells are also effective where the phreatic line is high and it becomes necessary to

lower it. In both of these instances, the effect of the wells should be checked by piezometers for both sequence of pumping and the adequacy of the number of wells. The permeability of the foundation soils will have a great effect on this drawdown produced by the wells.

The relief wells should have a minimum diameter of 6 inches or larger, depending on the flow expected. The screens or perforations should be surrounded by filter material to insure the free flow of water into the pumps. The water from these wells should be monitored for metallic ions, both as an environmental concern and to guard against or anticipate corrosion of the pumps. The U.S Army Corps of Engineers has published material on relief well design (46).

## Vertical Sand Drains

Vertical sand drains can be used in special situations and would be located in the same area as the relief wells relative to the downstream portion of the dam. Such a situation might be where a tailings pond was to be built on top of a thick layer of compressible soils with a water-bearing aquifer below under artesian head. A series of sand drains approximately 1 foot in diameter would be drilled through the soil and into the aquifer and filled with sand to the surface, where they connect to a blanket drain to the downstream toe. This would relieve the pressure on the aquifer as the dam increased in height so that the foundation could settle at a faster rate without undue uplift. As previously stated, each area is different and must be treated as the field conditions dictate.

## Rate of Seepage

The rate of seepage through a tailings embankment is governed by the permeability of the materials, the hydraulic gradient, and the elevation difference between the pond and the point of emergence of the seepage. With a coarse grind, low pulp density, and a wide beach, the seepage vertically through the beach is tremendous and could equal or exceed that which escapes through the embankment. With a given screen analysis, mineralogy, tonnage, pulp density, area, and construction method (upstream or downstream), there is little that can be done to change the seepage once the operation has started. The length of the flow path cannot be increased after the pond gets to the extreme upstream position, or to the decant, except by adding material to the toe. The difference in elevation has to increase, and no impervious barriers can be placed. Therefore, it is imperative that seepage be anticipated and incorporated into the design. The mathematical expression for the rate of seepage is

$$q = k \frac{n_f}{n_d} h, \tag{3}$$

where $q$ = the rate of seepage per unit length perpendicular to the plane of the flow net,

$k$ = the coefficient of permeability of the soil.

$n_f$ = the number of flowpaths (determined from the flow net),

$n_d$ = the number of equipotential drops (determined from the flow net),

and $h$ = the difference in piezometric head between the point of seepage entry and the point of seepage exit.

An approximate coefficient of permeability can be determined by laboratory tests if field tests are not possible. For more details see appendix C and Cedergren (19).

## Surface Runoff Control

Methods of estimating runoff are described in appendix B. Tailings embankments that are built in or across a major drainage area must have some method of controlling the runoff into the pond or be able to contain the runoff from a major (100-year) flood. Generally, a decant system or barge pump cannot handle this type of flow unless it is specifically designed for that flow. In mountainous areas, a site of any major size would have several square miles of drainage area upstream from the tailings pond. This would require facilities to be designed to handle this flow. Some design measures could include:

1. The tailings pond can be built across the entire valley drainage with a dam confining the tailings on both the downstream and upstream sides. A large culvert through the entire length of the embankment would be designed to easily handle the normal spring flood. In a 100-year flood, the entire tailings embankment would act as a dam for temporary storage of anything that exceeds the capacity of the culvert.

2. The cross-valley embankment can be designed with extra-large decants, maintaining sufficient freeboard to contain the flow above the capacity of the decant. The decants must be of sufficient size to assure that stored water from one storm is drained within a reasonable time so that runoff from a subsequent storm can be contained. The embankment must be designed for the maximum water surface. Barge pumps alone would be unsafe because a power failure during such a flood would have to be assumed.

3. The major portion of the runoff can be diverted into a diversion channel and bypass the embankment entirely. This is very difficult to do in steep mountainous terrain, because the diversion ditches are very likely to silt up and overflow during a major storm.

4. Spillways can be built after the embankment is abandoned, but are not feasible while the embankment is being raised.

5. In desert areas where flash floods are very common, diversion ditches are easier to maintain and are a logical protective measure.

Determining the freeboard required at any time to store flood runoff will involve calculating the pond capacity from the topographic maps. Storage capacity during the first few years is most critical because of the small size of the pond, so construction schedules must be maintained to keep sufficient freeboard.

Hydraulic handbooks describe the construction of diversion ditches and spillways. There are several critical points of concern with diversion ditches. One is that they be open and remain open in time of major floods; silting in and overtopping into the tailings area is the main difficulty that makes them inoperable when they are needed. The second is that they have the required capacity and a relatively flat gradient so that erosion does not occur near the embankment. Channels can be protected against erosion by concrete lining or paving stones.

## INITIAL CONSTRUCTION

### Site Preparation

The foundation investigation and sampling will dictate what has to be done to prepare for dam construction; site preparation will vary considerably depending on whether the dam is to be high or low (<100 feet high) and whether it is to be a true water-type dam or not. If it is to be a true water-type dam, consultants familiar with dam construction are a necessity. Therefore, water-type dams are not covered in this discussion.

If the tailings dam is to be on or near bedrock which is relatively impervious, an inspection of the bedrock may be warranted to check for open fissures that must be sealed to prevent piping. Coarse foundation soils and buried coarse talus should be removed. Excavation of all vegetation, surface growth, pockets of peat, and zones of weak and pervious soil should be performed, resulting in competent foundation material. Consolidation testing may be necessary and foundation scarifying and compaction of foundation soil may be required to attain a sufficiently strong foundation.

Where the entire tailings area is on deep alluvium with a permeability (K) of $10^{-2}$ to $10^{-3}$ centimeters per second, the seepage cannot be stopped by the dam because most of the seepage water goes through the subsoil and not through the base of the dam. A cutoff trench is sometimes used in the construction of a starter dam where conditions warrant its use, such as a pervious foundation extending to a shallow depth. The cutoff trench can intersect a relatively impervious layer to reduce the downstream seepage. A cutoff trench may also be used where the foundation is on bedrock and a cutoff and anchor are needed. The cutoff trench would more probably be used with the centerline or downstream method where seepage through the starter dam is not wanted. It would not be used where a blanket or strip (gravel) drain was to be used extending upstream from and completely beneath the starter dam, but it could be used where pipe drains extend through the starter dam.

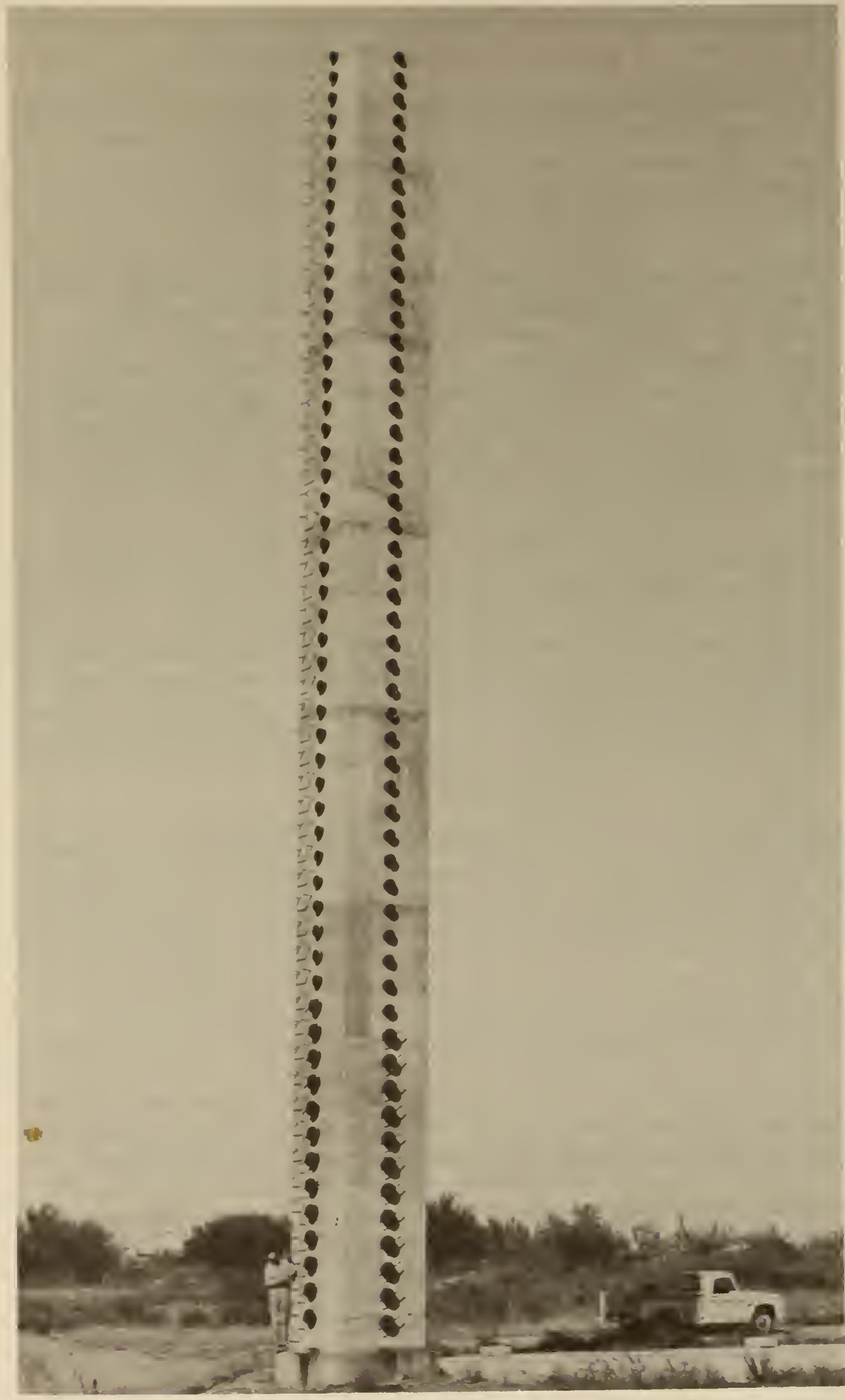

FIGURE 17. - Decant tower with 6-inch outlets at 4-1/2 inches center-to-center.

## Water Reclaim Systems

### Decant Pipe and Towers

The most common method of reclaiming water from a tailings pond is through decant tower and lines (figs. 17-18). These can vary from a simple 8-inch pipeline laid along the ground from the downstream toe to the clear water area and extended as the dam is raised, to large steel and reinforced-concrete conduits with reinforced-concrete towers. The former is used for small operations, and the latter is designed for a 500-foot-high embankment (figs. 17-19). In this system the clear water near the surface of the pond flows through closely spaced openings on top of the pipe or in the tower through the conduit under the starter dam to waste or to a holding pond where it is pumped back to the concentrator water storage pond. The openings in

FIGURE 18. - Decant line to tower 6-inch outlets at each 6-inch elevation rise.

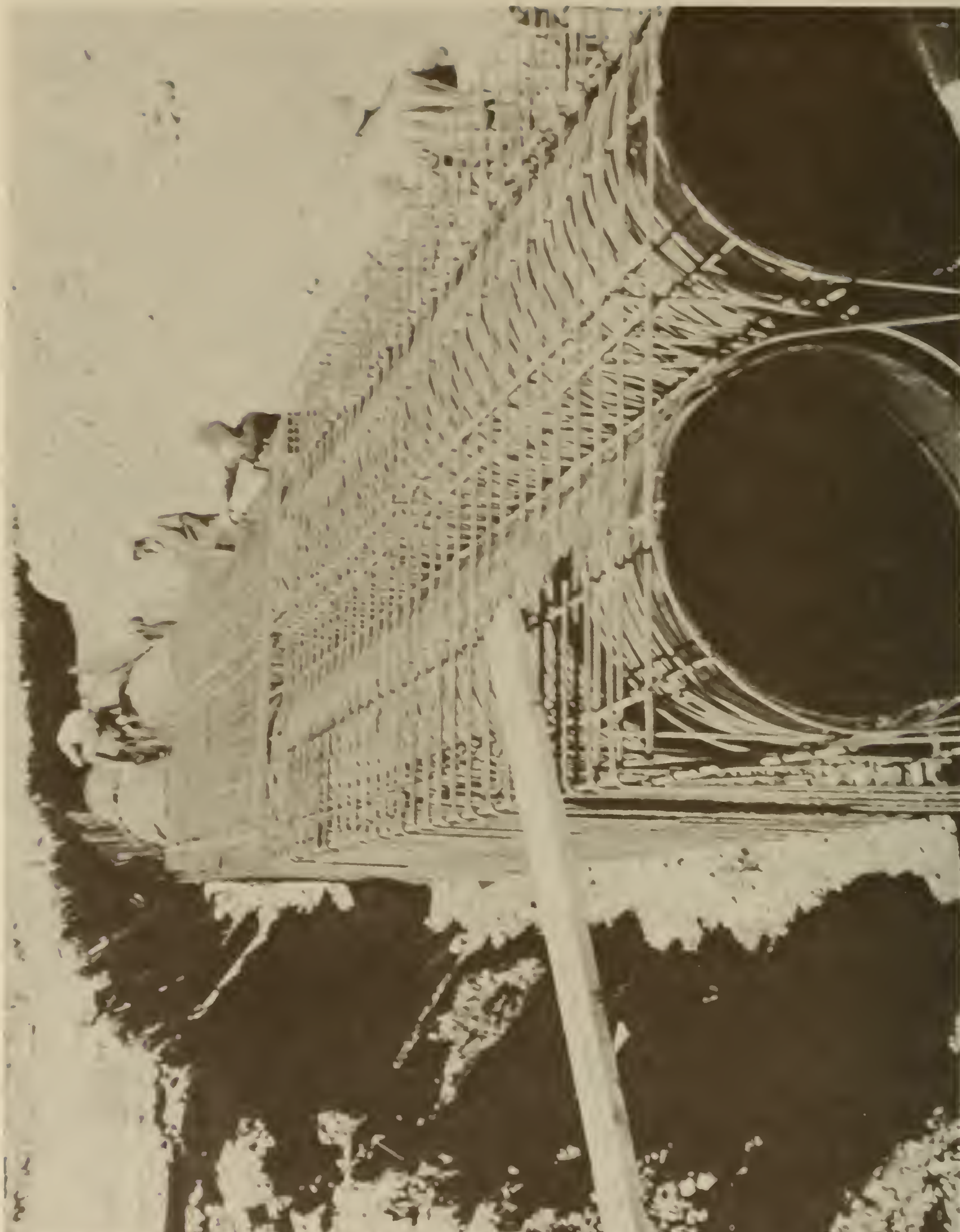

FIGURE 19. - Steel-reinforced decant lines—42-inch ID.

the tower are as close as 4-1/2 inches center-to-center (fig. 17) in order to have very close control of the water pond level and are closed off by metal caps as the slime approaches this level. Figure 20 shows a schematic of a decant line and tower, a barge pump, a siphon, and a pump with a flooded intake. The latter takes advantage of a high positive suction head, which is not available with the ordinary decant line and water storage pond. The head differential would be as great as the height of the pond, which is the reason barge pumps are economical. They also take advantage of the lower head on the return waterline. They do not, however, lend themselves to close control of the pond and can require a second compartment for emergency overflow. In flat

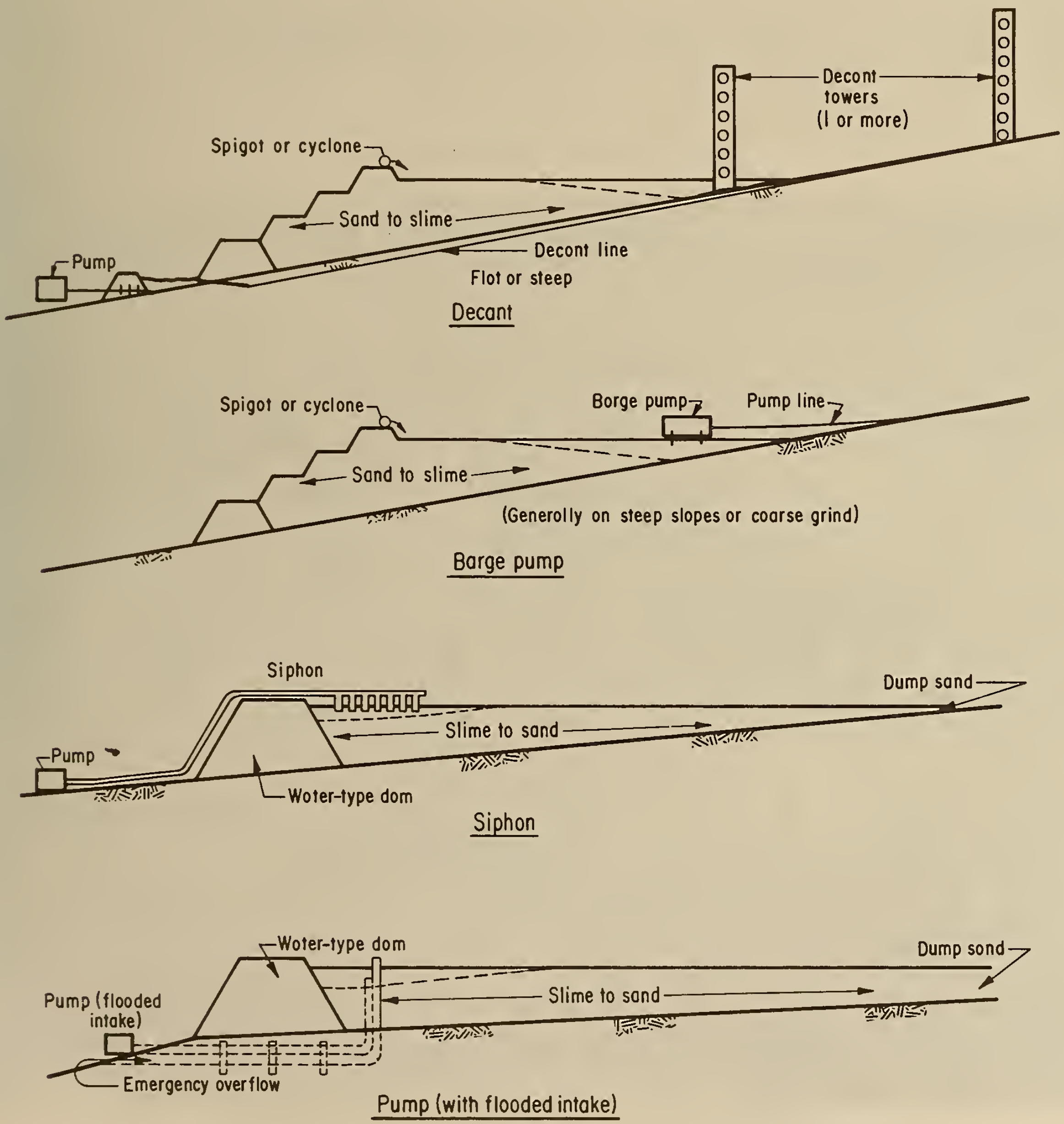


FIGURE 20. - Schematic of decant, barge pump, siphon, and pump with flooded intake.

country only one or two towers may be required for a pond with 1-square-mile area. The two towers serve as a backup for emergency or malfunction. In a valley-type deposit where the terrain may have variable slopes, low towers should be built at appropriate elevations up the valley so that two towers are in operation at all times. When these towers are abandoned, they should be blocked off with concrete. For extra safety, both decant towers and barge pumps can be combined, utilizing the best parts of both systems (fig. 21). The intake to the decant pipe should be protected from trash if necessary.

## Barge Pumps

This method of reclaiming water from a tailings pond is becoming more popular because of its versatility and lower cost, especially in the larger operations where high dams are planned. The cost of long decant pipes of heavily reinforced concrete may be many times the cost of a barge and pump. The barge pump gains considerable static head over the decant lines with pumps. This results in a reduction in required power and cost (figs. 22-23).

FIGURE 21. - Barge pump and decant tower in the same pond.

FIGURE 22. - Barge pump and line—steep terrain.

FIGURE 23. - Barge pump and line—iron ore mine. *(Courtesy, Erie Mining Co., Minnesota.)*

The main disadvantage of this system is that there is no way to take care of a sudden flood when the pond fills and the flow is beyond the capacity of the pumps. A power failure during flood stage would also be very serious. A study of the maximum 100-year flood must be made to see if enough freeboard is available to take this amount of water without overtopping. Where there is no watershed above the pond, or where flash floods can be diverted, this is no problem.

Where a tailings area has two or more canyons in which clear water collects, the mobile pump on a rubber-tired trailer can be used in place of a barge pump.

## Siphons

A third system uses siphon pipes installed over the crest of the embankment to draw water from the pond and discharge it to the downstream toe of the embankment to a holding pond (fig. 24). This system should be used only on a water-type dam where the water can be up against the dam. Dams built of tailings sand or borrow material should not have the water as high or as near to the embankment as is necessary for the siphon. In special situations, where seepage is not a problem and coarse waste is available for downstream protection, the siphon can be used.

## Comparison of the Three Systems

### Decant System

The system has the following advantages:

1. Mechanical and electrical failures do not stop discharge from the pond.

2. The operation is extremely simple.

3. If decants are designed with sufficient capacity, they can serve as permanent drains and handle runoff to keep the pond empty, or maintain a constant pond elevation after the tailings operation has been abandoned.

FIGURE 24. - Siphon on large tailings pond. *(Courtesy, Kennecott Copper Co., Utah.)*

The system has the following disadvantages:

1. The pumping head is higher if water is collected at the downstream toe of the embankment and pumped up to the mill.

2. High decant towers are susceptible to wind damage and are also susceptible to damage by tailings solids surrounding them. This is especially true where tailings are dumped into the pond at various places and might slump in a large mass against the tower. There is also danger from ice damage in cold climate. If spigoting along the crest of the embankment is the only method of discharge, there is less danger of damage.

3. The decant lines and towers must be designed to withstand the full hydrostatic pressure of saturated tailings to prevent failure.

4. Foundation settlements are likely to crack or open joints in decant culverts, leading to piping into and through the culvert. For this reason monolithic reinforced-concrete culverts are preferred over precast concrete sections.

5. Pipes that have collapsed or cracked are nearly impossible to repair, and leaks are almost impossible to stop.

6. Culverts and towers are more expensive to construct than barge pumps.

<u>Barge Pump System</u>

The system has the following advantages:

1. It is easy to operate in the cross-valley embankment where the terrain is steep, the grind is relatively coarse, and the clear water pool is deep.

2. The power consumption is less than for the decant system.

3. The cost of a barge and pump is much less than that of a decant system for large-tonnage operations.

The disadvantages are--

1. Barge pumps cannot be used in relatively flat terrain with a fine grind (keeping them out of the mud becomes a problem).

2. Pumps must be raised periodically as the pond rises.

3. Freezing is a problem in cold climates. (Low-pressure air bubbling from submerged pipes can keep the barge free of ice.)

4. Pumps cannot be designed to handle the 100-year flood, so enough freeboard must be provided for this emergency.

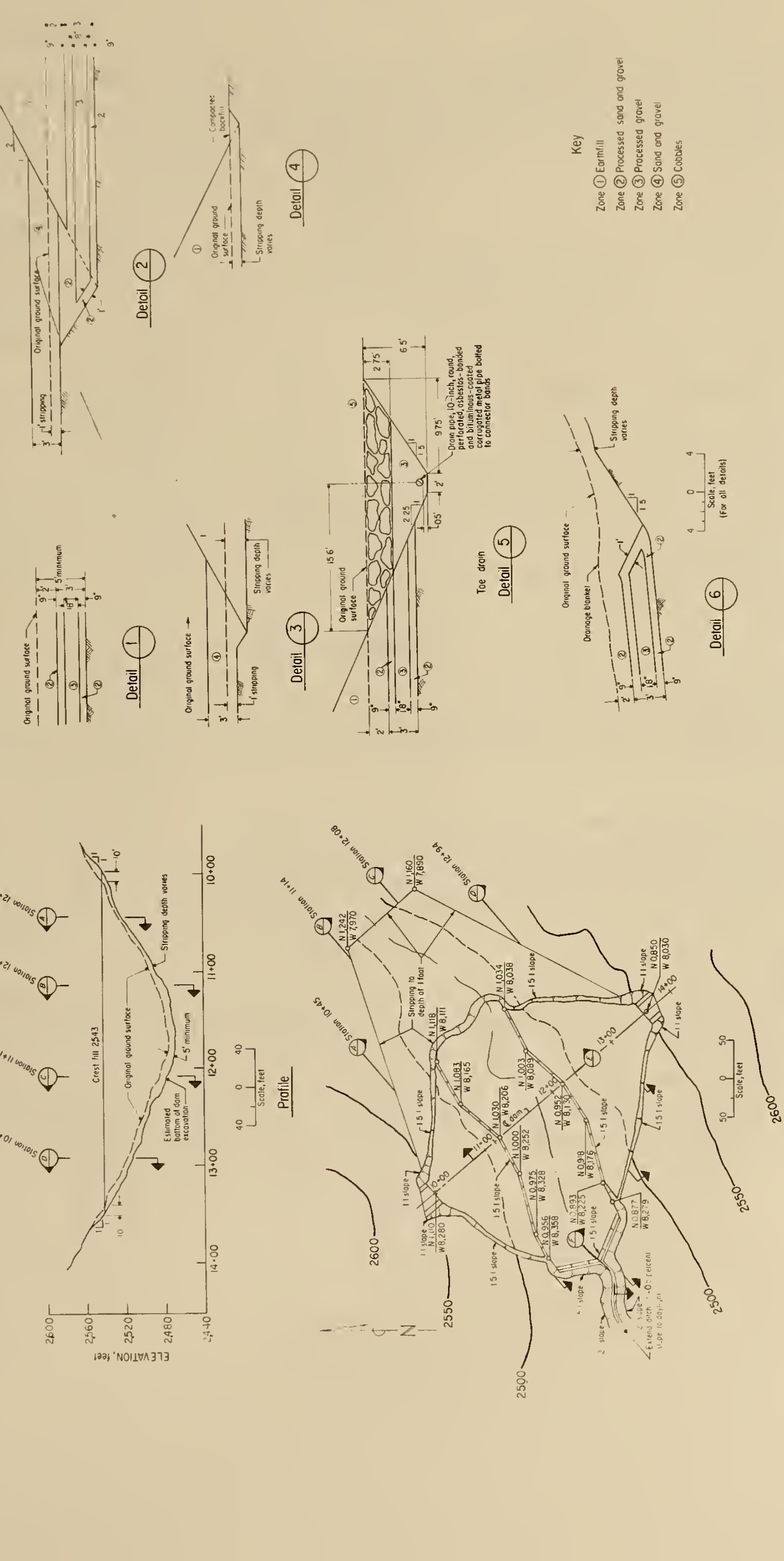


FIGURE 25. - Starter dam excavation—general plant section.

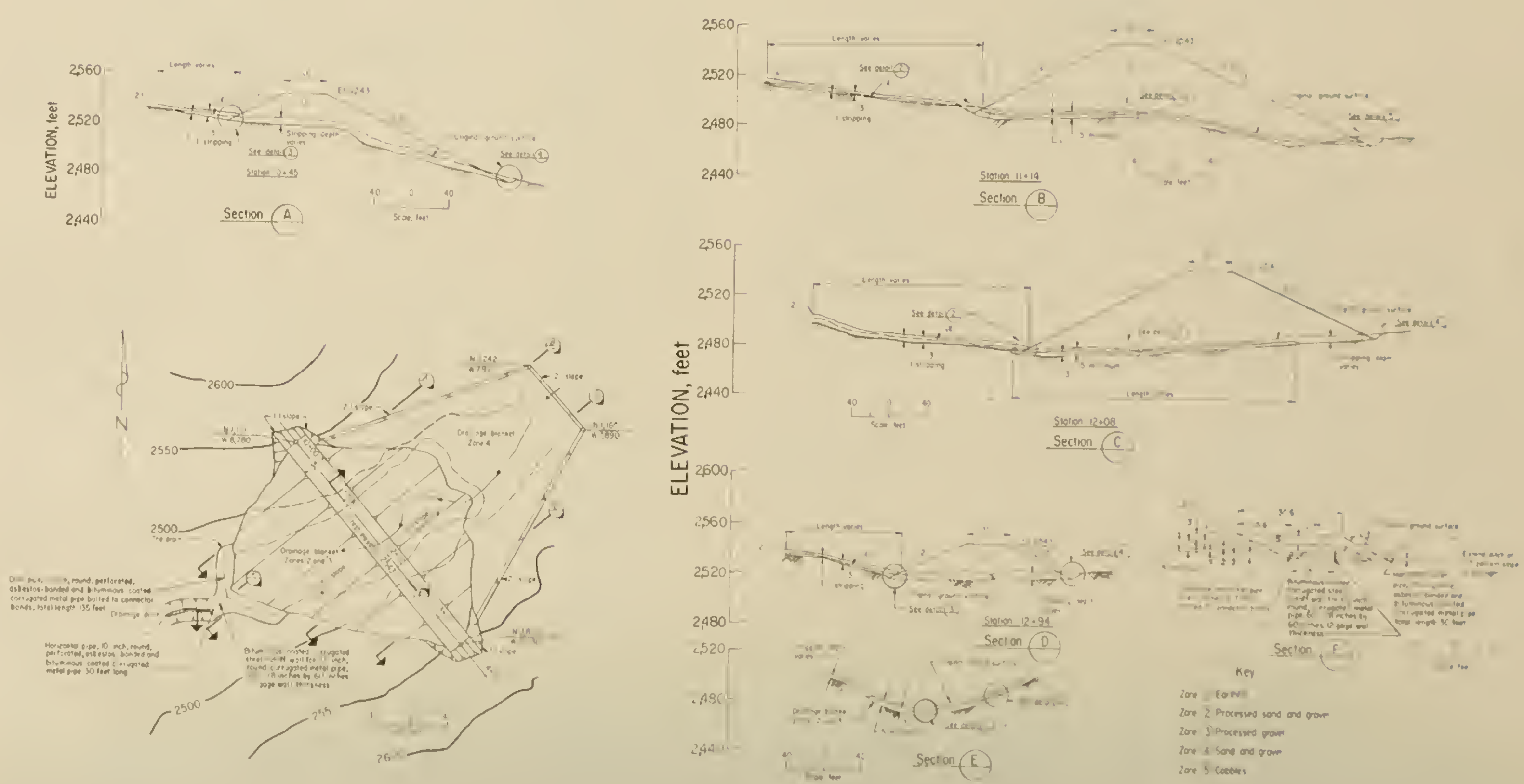


FIGURE 26. - Starter dam construction—detail.

Siphon Reclaim Systems

These systems are not recommended for tailings embankments for the reasons already stated, but where one can be used, it has the advantage of being able to pass full-capacity discharges with narrow limits of water surface rise in the pond.

It has other disadvantages:

1. Embankment cracking because of settlement can be a danger.

2. It cannot pass ice or debris.

3. Water can freeze in the inlet legs and air vents before the water rises enough to prime the siphon.

4. Siphon pipes are subject to cavitation and low absolute pressures, and so are limited to a total drop of 15 feet on earth dams.

5. The make-and-break acting of the siphon causes surges and stoppages in flow.

## Starter Dam Construction

When the dam site excavation is complete and decant lines and drains have been constructed through the base of the dam, the dam construction itself can proceed.

The excavating and hauling of material from the various borrow areas must be closely supervised so that each zone in the starter dam receives the proper material, the layers are placed on the dam in proper thickness, and the moisture and compaction are up to specifications. Moisture and density samples must be taken frequently to insure proper density.

It has been stated previously, but it cannot be overemphasized, that the starter dam using the upstream method construction should be relatively permeable, whereas with the downstream method it should be relatively impermeable. See figures 25 and 26 for typical detail of starter dam construction. Each area has its distinct problems, and these figures merely illustrate some of the detail necessary for proper construction. Figure 27 shows the screen analyses of the four zone materials.

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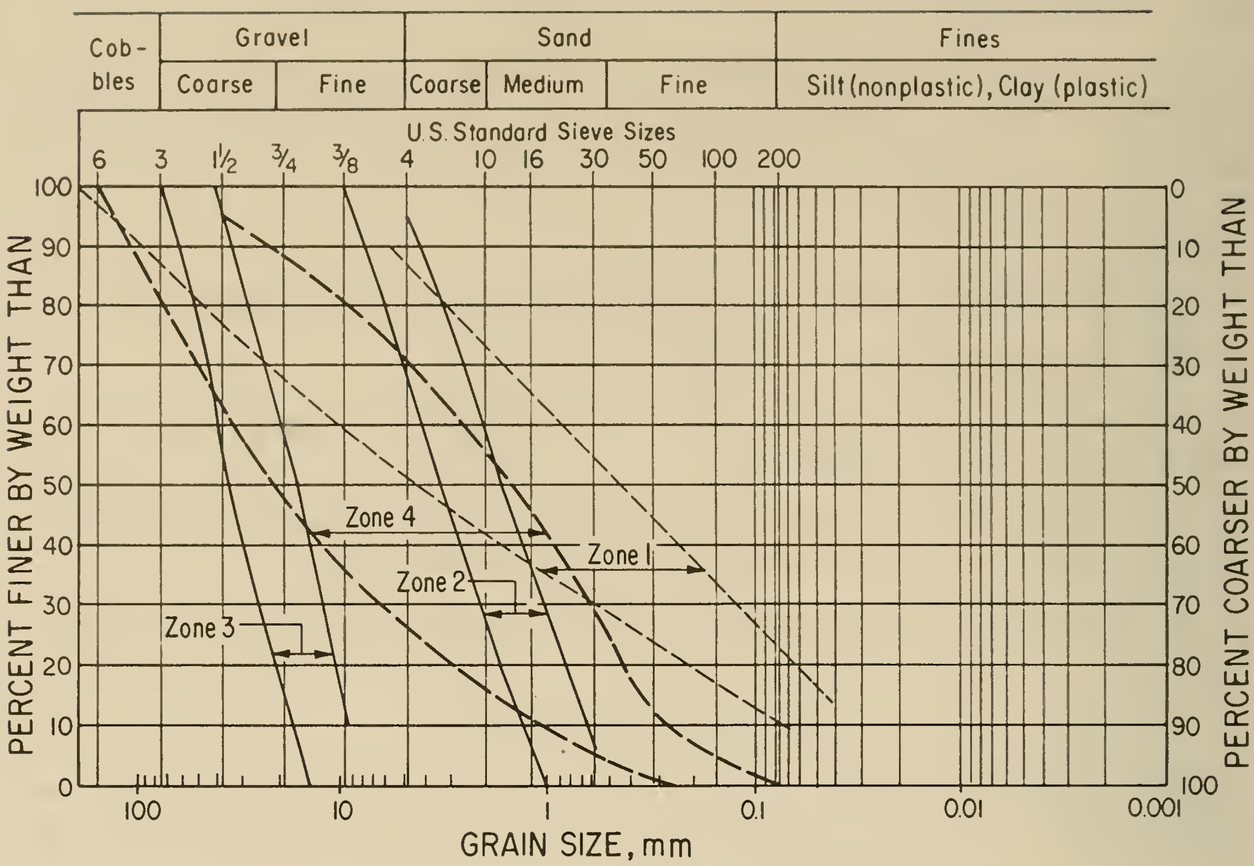


FIGURE 27. - Screen of four zone materials.

## Fill Compaction

Compaction of a soil is usually accomplished by spreading the soil in layers of specified thickness and compacting with a mechanical compactor. A number of arbitrary standards for determining the optimum moistures and maximum densities have been established to simulate different amounts of effort as applied by the full-sized equipment used in soil construction. The simplest and the most widely used are the Standard Proctor (6) and the Modified Proctor (9). Compaction may be specified by procedure (type of compactor, layer thickness, number of passes, and moisture content to be used) or by end product (minimum inplace density required). The purpose of compaction may be to increase the fill shear strength or to decrease the fill permeability or both.

Variables affecting compaction are--

1. The type of compactor: Pneumatic-tired roller, steel wheel, vibratory steel wheel, sheepsfoot, grid roller, vibratory plate compactors, and track-type tractors.

2. The weight and energy of the compactor.

3. The thickness of layers.

4. The placement water content.

Table 5 shows the compactors recommended to be used to attain 95 to 100 percent of the Standard Proctor on various soils, but during construction regular density testing is necessary. For clean cohesionless tailings sand, compaction in thin lifts with proper moisture might be attained with a tractor for building dikes, but it should be checked with inplace density tests. Seldom does the sand have less than 8 percent minus 200-mesh material, and generally the moisture content is in the bulking range where compaction is difficult.

The pneumatic tire has proved to be an excellent compactor for cohesionless and low-cohesion soils, including gravels, sands, clayey sands, silty sands, and even sandy clay. It applies a moderate pressure to a relatively wide area so that enough bearing capacity is developed to support the pressure without failure. The light rollers are capable of compacting soils in 4-inch layers to densities approaching the Standard Proctor maximum at optimum moisture with three or four passes. The heavy rollers can obtain densities above Standard Proctor maximum in layers up to 18 inches thick with four to six passes.

Because of the small loaded area and high unit pressure, the sheepsfoot roller is adapted best to cohesive soils such as clays. Tailings embankments with more than 20 percent minus 200-mesh material in the dike could be compacted with the sheepsfoot or the modified sheepsfoot, which has some of the feet removed and flat plates 8 to 10 inches in diameter welded on the remaining feet. This modified version is best for silty soils of low cohesion, which better describes some of the tailings sand. Moisture control is very important to efficiency of compaction of this type of material.

For cohesionless borrow and waste materials, such as rockfills and gravels, compaction by track-type tractors and haulage trucks might be adequate if properly controlled. Where additional compaction is required, heavy vibratory steel wheel compactors are very efficient and are not moisture dependent.

Compaction is very critical on starter dams whether pervious or impervious and on tailings embankments and dams built of tailings sand to raise density, increase stability, and prevent liquefaction.

A particular compactor has its own optimum water content which may not be the same as that determined in the Standard or Modified Proctor test in the laboratory. The laboratory tests will be within a few percentage points of those applicable to most compactors, and the heavier the compactor, the lower will be the optimum moisture content. Field density tests will help determine the efficiency of a particular method. Use of compaction layers which are too thick for a particular compactor will result in undercompaction at the base of the layers, with increased horizontal permeability of that zone.

TABLE - Compaction equipment and methods

| Uses | Requirements for compaction to 95 to 100 percent Standard Proctor maximum density: Compacted lift thickness, inch | Requirements for compaction to 95 to 100 percent Standard Proctor maximum density: Passes or coverages | Requirements for compaction to 95 to 100 percent Standard Proctor maximum density: Dimensions and weight of equipment | Possible variations in equipment |
|---|---|---|---|---|
| **SHEEPSFOOT ROLLERS** | | | | |
| For fine-grained soils or dirty coarse-grained soils with more than 20 percent passing the No. 200 sieve. Not suitable for clean coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important. | 6 | 4 to 6 passes for fine-grained soil.<br>6 to 8 passes for coarse-grained soil. | Soil type / Foot contact area, sq in / Foot contact pressures, psi:<br>Fine-grained soil, plasticity index, $>30$. / 5-12 / 250-500<br>Fine-grained soil, plasticity index, $<30$. / 7-14 / 200-400<br>Coarse-grained soil. / 10-14 / 150-250<br>Efficient compaction of soils wetter than optimum requires less contact pressure than for the same soils at lower moisture contents. | For earth dam, highway, and airfield work, drum of 60-inch diameter loaded to 1.5 to 3 tons per lineal foot of drum generally is utilized. For smaller projects, 40-inch-diameter drum loaded to 0.75 to 1.75 tons per lineal foot of drum is used. Foot contact pressure should be regulated to avoid shearing the soil on the 3d or 4th pass. |
| **RUBBER-TIRE ROLLERS** | | | | |
| For clean, coarse-grained soils with 4 to 8 percent passing the No. 200 sieve. | 10 | 3 to 5 coverages. | Tire inflation pressures of 60 to 80 psi for clean granular material or base course and subgrade compaction. | Wide variety of rubber-tire compaction equipment is available. For cohesive soils, light-wheel loads, such as provided by wobble-wheel equipment, may be substituted for heavy-wheel load if lift thickness is decreased. For cohesionless soils, large-size tires are desirable to avoid shear and rutting. |
| For fine-grained soils or well-graded, dirty coarse-grained soils with more than 8 percent passing the No. 200 sieve. | 6- 8 | 4 to 6 coverages. | Wheel load 18,000 to 25,000 pounds. Tire inflation pressures in excess of 65 psi for fine-grained soils of high plasticity. For uniform clean sands or silty fine sands, use large-size tires with pressures of 40 to 50 psi. | |
| **SMOOTH-WHEEL ROLLERS** | | | | |
| Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures. | 8-12 | 4 coverages. | Tandem-type rollers for base course or subgrade compaction. 10- to 15-ton weight, 300 to 500 pounds per lineal inch of width of rear roller. | 3-wheel rollers are obtainable in wide range of sizes. 2-wheel tandem rollers are available from 1 to 20 tons. 3-axle tandem rollers are generally used in the range of 10 to 20 tons. Very heavy rollers are used for proof rolling of subgrade or base course. |
| May be used for fine-grained soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands. | 6- 8 | 6 coverages. | 3-wheel roller for compaction of fine-grained soil; weights from 5 to 6 tons for materials of low plasticity to 10 tons for materials of high plasticity. | |
| **VIBRATING-BASEPLATE COMPACTORS** | | | | |
| For coarse-grained soils with less than about 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing No. 200, placed thoroughly wet. | 8-10 | 3 coverages | Single pads or plates should weigh no less than 200 pounds. May be used in tandem where working space is available. For clean coarse-grained soil, vibration frequency should be no less than 1,600 cycles per minute. | Vibrating pads or plates are available, hand-propelled or self-propelled, single or in gangs, with width of coverage from 1-1/2 to 15 feet. Various types of vibrating-drum equipment should be considered for compaction in large areas. |
| **CRAWLER-TRACTOR** | | | | |
| Best suited for coarse-grained soils with less than 4 to 8 percent passing No. 200 sieve, placed thoroughly wet. | 10-12 | 3 to 4 coverages. | No smaller than D8 tractor with blade, 34,500-pound weight, for high compaction. | Tractor weights up to 60,000 pounds. |
| **POWER TAMPER OR RAMMER** | | | | |
| For difficult access, trench backfill. Suitable for all inorganic soils. | 4-6 for silt or clay.<br>6 for coarse-grained soils. | 2 coverages. | 30-pound minimum weight. Considerable range is tolerable, depending on materials and conditions. | Weights up to 250 pounds. Foot diameter 4 to 10 inches. |

No one method of compaction can be established as best for the sand zones of tailings embankments because of the wide variations in the gradation of the sand on the beach from one property to another. Each mine must determine the most efficient compactor for the materials it is using.

## CONSTRUCTION DURING OPERATION

### Coarse Sand and Clean Separation by Spigoting

The beach formed from tailings containing 38 to 40 percent minus 200-mesh material discharged at 30 percent pulp density (fig. 9) is a relatively clean sand with 10 to 15 percent minus 200 mesh and makes a good dike-building material. It will drain rapidly and can be moved with a dragline or dozer from the beach to build the dike when the moisture content is optimum for good compaction. Tests should be made on this material to determine the optimum moisture, depth of each layer to be compacted, and method of compaction. Care should be taken that the moisture of the sand does not get into the bulking range where it is virtually impossible to get good density. See table 5 for compaction equipment, moisture, and compaction lift required to attain 95 to 100 percent of Standard Proctor. The permeability of this beach material can be in the range of $1 \times 10^{-2}$ to $1 \times 10^{-3}$ centimeters per second.

### Fine Grind and Poor Separation by Spigoting

The beach formed from tailings containing 55 to 60 percent minus 200-mesh material and discharged at 48 to 50 percent pulp density (fig. 10) has very little separation of the sand and slime on the beach, has essentially the same screen analysis as the general mill tail, and results in a very poor dike-building material. This type of material should probably be cycloned or spigoted at a much lower pulp density to make a cleaner sand. This material will have a permeability of $1 \times 10^{-5}$ to $1 \times 10^{-6}$ centimeters per second and will be very slow to drain. In this case it would be very difficult to achieve the required density to stabilize the fill. These examples probably represent the two extremes of material encountered in U.S. metal mines where the tailings are used for building dikes.

The output from a cyclone might have variable grain-size gradation depending on the raw material. If the sand from the cyclones has a very high slime and moisture content, it may flow at very flat slopes and go beyond the design boundaries of the embankment. It will be slow to drain and difficult to handle. If slime content is low, the sand will drain rapidly and be ready for spreading and compaction by mechanical equipment soon after deposition.

If the minus 200-mesh material contains clay minerals, the permeability of the cyclone underflow would be lower than if these fines were in the coarser silt sizes. For this reason, a hydrometer analysis determining grain-size gradation in the minus 200-mesh material will help determine the handling and permeability characteristics of the sands on the embankment.

Carefully controlled cyclones can produce a very uniform product, but when they are on a tailings embankment with all the variables there is a great

difference in the product. The pulp density, feed rates, pressure, and wear on the cyclone orifice all make a difference in the cyclone underflow, and there is little that can be done on the short term to change the cyclone adjustment to compensate for it.

The gradation of the tailings from the mill is entirely dependent on the grind necessary to free the ore minerals from the gangue. This is determined first in the laboratory and then in a pilot mill during the design phase of a new mine. When a suitable grind has been determined in the pilot mill, tests can be made to determine the types and sizes of cyclones and the number of stages necessary to provide a suitable underflow. Spigoting tests of the sands can also be made to simulate the segregation on the beach to determine if this method can be used. From this same material a probable range of permeabilities of the sand can be determined and will enable the designer to incorporate suitable seepage control provisions into the design. The tailings produced by the cyclones may be adjusted during early stages of operation to get the proper sand for embankment construction. The sand separation and placement should be carefully watched. An attempt to recover additional metals could make a change in the mill circuit and also affect the tailings pond.

## Sand Yield

The yield of suitable sand obtained in separating the coarser fraction from the raw tailings affects the design and construction of the embankments. A cross-valley dam has a big advantage over a flat-country impoundment where three to four sides have to be built using sand. A shortage of sand makes it difficult to keep the embankment crest above the finer tailings in the pond. With fine grind (55 to 60 percent minus 200 mesh and finer) and in flat country, cyclones with the upstream method may be one way to keep the embankment ahead of the tailings. In this case it may be necessary to use borrow or mine waste as a supplement to the sand.

Operations with 35 or more acres per 1,000 tons of daily capacity, with this fine grind and with tailings areas divided into two separate ponds that are used alternatively, can operate with peripheral discharge if the ultimate dams are kept low and the tailings are allowed to drain and consolidate after each 10±-foot lift.

The yield of acceptable sand from cyclones can be calculated from the gradation of the raw tailings and the characteristics of the cyclones. The rate of embankment construction will depend on the amount of available sand, the length of embankment being built, and the weather, or the number of months a year that it is possible to construct embankment. Using cyclones and the downstream method, each foot of rise takes longer and requires more sand than the previous foot. The use of cyclones with the centerline method is nearly as bad.

Spigoting and building an upstream embankment with the beach sand requires the same amount of sand each lift except for the increased length as the embankment gets higher.

In planning any tailings site, the active time for embankment utilization is far below 100 percent. The time required to build embankment and replace spigots or cyclones and the time necessary to raise the entire line to a new berm are times when the pond is not available for discharging tailings unless they can be "dumped" at some other spot in the pond. For this reason, it is better to have two complete and separate dams. This is especially important at the start of a new operation. With two dams there can be a complete shut-down of an area so that the sand beach can be drained, dike built, and pipes or cyclones replaced. By alternating sites, a regular schedule of maintenance and operation can be set up; also the annual rise of the embankment is reduced, which improves slope stability. Where the winters are severe, dike building can be done only in the 6 to 8 warmer months to prevent the formation of ice lenses in the beach area. Enough sand must be available to build enough dike in the summer to last through the winter months. Where tailings sand is used for mine stope fill, the amount of sand available for embankment construction is further reduced; of course, the total volume to be impounded is also reduced by this amount.

When the grind is such that the proportion of minus 200-mesh tailings is more than 55 to 60 percent, the use of cyclones is almost mandatory in order to save the entire volume of sand for dam building. Under certain conditions, a water-type dam should be considered even though the capital cost is high. For these dams the operating cost is very low. Conditions that may warrant water-type dams are high percentage of slimes, harmful chemicals in the tailings, and, for phosphate clay, slimes with no sand in the tailings.

Tailings embankments that are constructed predominantly of sand recovered from mill tailings slurry can be constructed by one of the four basic methods described on pages 65-71.

## Upstream Construction With Spigot Sand

The common method in spigoting is to lay the header pipe on the top of the completed starter dam along the upstream edge with the spigot valves pointed upward at about 30° angle (fig. 28). The spacing of the valves along the header pipe depends on the total tonnage and the size of the spigot pipe, ranging from 10 to 15 feet for a 2-inch line and up to 50 feet for a 4- to 6-inch line. When the tailings area is filled to the top of the starter dam, the pipes are removed; the sand is allowed to drain and then is used to build a dam 8 to 10 feet high with a 2 or 3 to 1 slope, the downstream toe of which is very near the header pipe. The equipment used is generally a 1- or 2-cubic-yard dragline (fig. 29) and bulldozer or a bulldozer alone. The compaction of this construction should be 95 percent of Standard Proctor. The sand beach must be allowed time to drain before the sand can be moved, and this may be 1 to 2 months or more depending on the climate and the permeability of the material. For this reason another tailings area is needed to take care of production when a dam is being raised. The pipes are then put back in place up the slope of the dam and over the top to again fill the area. There should be enough head on the pipelines to raise the embankment 30 feet or more before it becomes necessary to build a new berm with enough room for a roadway, and the header pipe is then raised to the higher elevation. This

FIGURE 28. - Spigoting around periphery of dike—upstream method.

operation is more costly and more time consuming than the 10-foot lifts and is another nonuse time for the tailings pond. In planning a tailings disposal system, this nonuse time is very important to the overall schedule of operations and cannot be ignored.

## Upstream Construction With Cyclones

This method can be used where the grind is fine (60+ percent minus 200 mesh) and the dam has to be built on three or four sides. It requires much less sand than the downstrem method, and has good drainage through the sand with a phreatic surface generally following down the sand-slime contact. The sand must have a permeability of more than 100 times that of the slimes.

The procedure is the same as with spigoting except that the cyclones can be placed on towers 8 to 10 feet high or be mounted on movable trucks. The cyclone underflow goes directly on the upstream side of the starter dam and is allowed to spread at its natural angle of repose for that pulp density. The

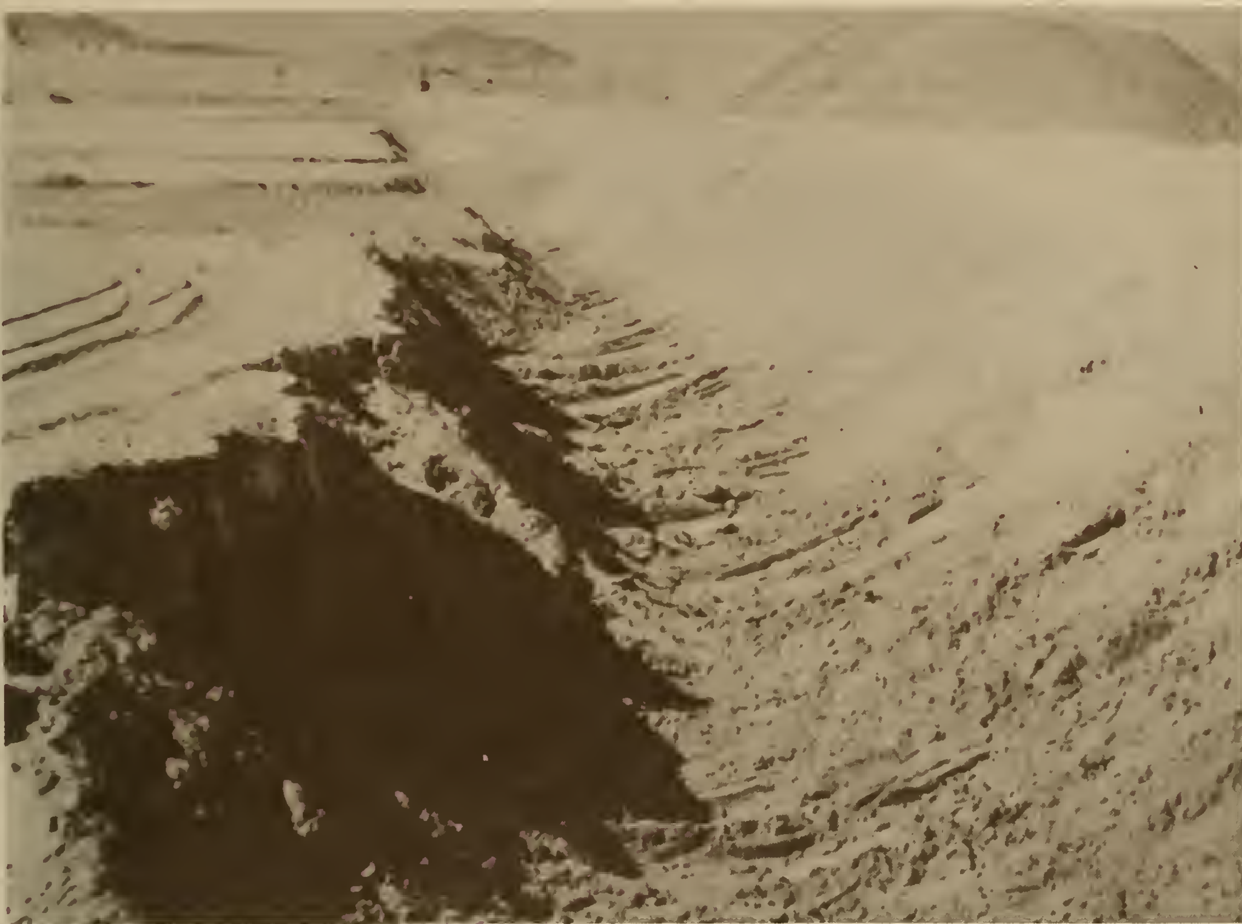

FIGURE 29. - Borrow pit after constructing dike with dragline—upstream method.
*(Courtesy, Pima Mining Co., Arizona.)*

overflow is piped 100 feet and farther upstream, and the two products flow together and have an irregular contact zone. The phreatic line generally intersects this contact zone, and the combination of a highly permeable sand against a slime zone has good drainage characteristics and keeps the sand quite dry.

The dam can be built to a height of 30 to 35 feet above the sand line as three successive cyclone towers are completely covered. The cyclones are then removed, the area is leveled to form a berm with room for an access road, the sand line is moved up to a new position, and the process is repeated (figs. 30-31).

With this method of depositing sand, it is very important that the starter dam be well protected by filters and drains along the upstream toe of the starter dam. The cyclone sand covers this drain and extends up the upstream face of the starter dam. This protects the starter dam from becoming saturated. The area immediately upstream from the drain will be covered with slimes which with time, drainage, and increased density will have a permeability as low as $10^{-6}$ or $10^{-7}$ centimeters per second or lower.

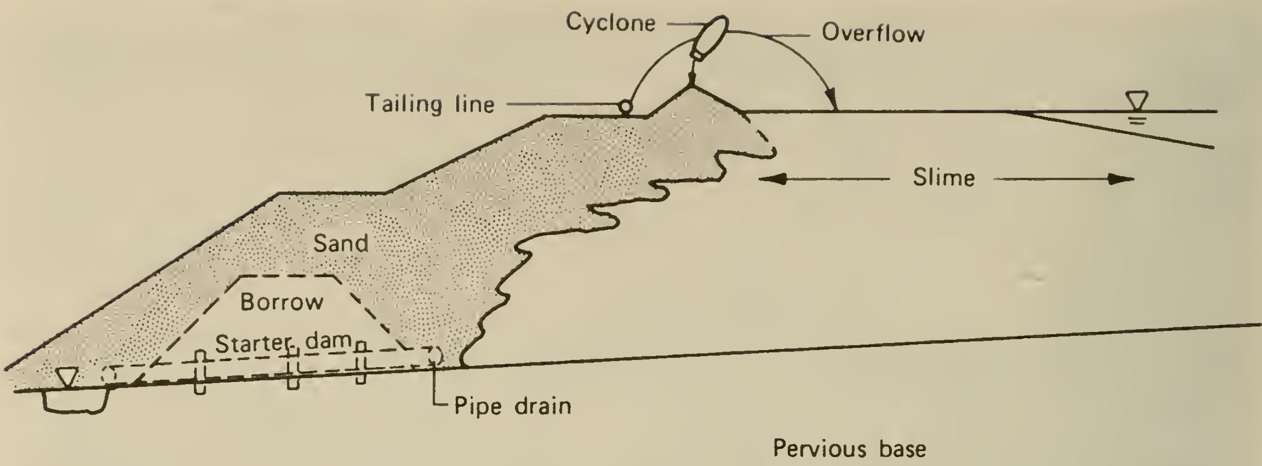


FIGURE 30. - Plan of upstream method with cyclones.

FIGURE 31. - Upstream method with cyclones. *(Courtesy, Magma Copper, Arizona.)*

To our knowledge, no research has been done and no stability analyses have been made on this type of deposit, but its stability may be as good as or better than that of dams built from spigoted sand of the same grind and pulp density.

## Downstream Construction With Cyclones

Only a few U.S. mines use a true downstream method with cyclones (fig. 32). This construction has been used in Chile where the mines are in highly seismic areas, in British Columbia where the grind is 60+ percent minus 200 mesh, in the new Missouri lead belt, and in White Pine, Mich. The main reason for this type of construction is that it is thought to be more stable and less susceptible to liquefaction under seismic shock.

The design and construction are quite varied in the different areas because of different conditions of terrain, tonnage, grind, etc. The starter dam is generally compacted borrow material and relatively impervious; a second downstream toe dam 200 to 300 feet below is pervious. Between the two dams

FIGURE 32. - Downstream method with cyclones. *(Courtesy, White Pine Copper, Michigan.)*

are drains--pipe, finger, or blanket--which drain to the outside of the toe dam to a holding pond.

The cyclones can be mounted on a truck which can start at one abutment 10 or more feet above the top of the starter dam; the overflow goes into the pond and the underflow is placed ahead of the cyclone truck and downstream of the starter dam. In this way the cyclone truck builds its own roadway in the direction of travel. Careful planning is required to assure room for the slimes while a berm lift is completed for the full length of the dam. This system has much merit and has these advantages:

1. The cyclones are in one place, and all have equal pressure and are easily available for maintenance.

2. Lifts as high as 30 feet can be made the full length of the dam in one pass, which reduces the moving time.

3. There is a reduction in manpower compared with other cyclone methods.

4. It utilizes more of the sand for dam building where the sand portion is low.

5. It apparently produces a good dam, although there have been no reports of stability analyses.

If the mobile cyclones are not used, the main tailings line can be laid along the middle or outside edge of the starter dam with the cyclones on towers 8 to 10 feet above the pipe and 10 to 15 feet downstream from the pipe. When the sand fills up to the underflow of the cyclones, new towers can be built farther from the pipe and the process repeated. If the area is large enough and the fines storage area is large enough relative to the dike length, as much as 30 to 35 feet of sand dike can be built before it is necessary to raise the main line. The system is awkward because it is difficult to have an access road on top of the dam with all the pipes and cyclones occupying the same area. If the dam can be built with this method faster than necessary for fines storage, the total tailings can be dumped for part of the year, which reduces the cost a great deal. This is especially beneficial in cold climates where dam building is impossible in the winter.

Covering the upstream face of the cycloned sand with compacted borrow should be unnecessary if a clean sand is produced which has a permeability of more than 100 times that of the slimes it is retaining. The slimes themselves form a beach against the sand which acts to reduce the amount of water flowing to the face of the sand. Because of the increase of permeability from the clear water pool to the sand, the phreatic line should drop rapidly to the drains between the two dams and out to the holding ponds. This system has the flexibility of building the dam rapidly and stays far in advance of the slime. If sand is short, borrow can be supplemented on the outside of the embankment. If pit waste were available, it could be used instead of borrow. If borrow is expensive, it should be avoided and alternative methods requiring less sand, such as the upstream method using cyclones, should be used.

If the embankment is constructed of tailings by the downstream method, a large tonnage of sand has to be placed downstream of the starter dam before the crest can be raised. At startup, if the starter dam is not high enough, the pond fills up with slimes before the dam crest can be raised by the cyclone underflow. A higher starter dam provides more time for placing sand on the downstream slope before the pond reaches the crest of the dam. This can also eliminate the need for borrow material to supplement the sand. If the starter dam is across a valley with a large drainage area, it should be designed to hold the runoff water as well as the normal mill water. The use of a temporary spillway to bypass runoff is really not feasible in most cases. The most suitable crest elevation will be determined by three factors--water classification, runoff control, and the most economical overall embankment cross section, which depends on quality and availability of building material.

The downstream method with cyclones and borrow is sometimes used where there may not be enough sand available by spigoting, or the segregation on the beach is not great enough to supply clean sand for dike building. Some are using it or advocating its use because of a higher FS where the sands are spread and compacted, achieving high shear strength. If a clean separation is attained by cyclones and the slimes form somewhat of a beach where deposited against the sand, the phreatic line can be kept low because of the much higher permeability of the sand. One disadvantage of this system is the additional cost associated with cyclones; another is the tremendous quantity of sand required as the dike gets higher. This method is inherently safer than the upstream method.

## Centerline Method

In the centerline method, the underflow from cyclones is deposited both upstream and downstream from the initial starter dike. The centerline of the ultimate dam is directly over the center of the initial starter dike. Earth-moving equipment can be used to flatten both slopes, or the slopes can be left at the angle of repose (fig. 33).

## Influences on Design

The four construction methods described in the preceding paragraphs result in substantially different embankment cross sections with different material characteristics and stability conditions.

In the upstream method of construction, the stability of the embankment is dependent largely on the shear strength characteristics of the tailings deposited upstream of the embankment. The embankment itself, built by dragline or bulldozer or both, will generally have greater shear strength and different permeability than the undisturbed material, but has little effect on overall stability because it is only a thin shell when viewed as part of a slip circle. The shear strength of the sedimented solids is governed by the gradation and density of the solids, the pulp density of the slurry, and the distribution of the pore water pressure within the deposit after sedimentation. The tailings as deposited have very low shear strength, but this increases with time as drainage and consolidation take place under the weight

FIGURE 33. - Cyclones on centerline method of dam building in small operation.
(Courtesy, St. Joe. Industries, Missouri.)

of the overlying material (fig. 4). Shear strength tests of representative samples of the tailings will give an indication of their effective angle of internal friction. Shear strength depends on two things, the friction angle ($\phi$ angle) and cohesion (C). The pore water pressure does not change the $\phi$ angle, but the buoyancy caused by water under pressure does decrease the shear strength. For this reason the pore water pressure within the deposit is important. Because of the large variations in permeability in different areas of the tailings deposit, it is difficult to predict the pore water pressures in advance. Tailings ponds of any appreciable size should be well monitored by piezometers to measure the pore water pressures as construction proceeds. If pore water pressures get too high, filling can be stopped and the area allowed to drain.

A computer analysis can be made of the tailings embankment at different water elevations to indicate at what level the stability becomes dangerous.

Because of the differences in waste minerals, gradation (fig. 2), and pulp density from one mine to another, the design of the tailings embankment will also vary greatly. The upstream method at a given mine can tolerate a certain downstream fill slope and be constructed to a given elevation and still have a 1.3 to 1.5 factor of safety. The actual maximum safe height will depend on the shear strength of the material. To increase the height to which the tailings dam can be constructed, the downstream slope must be flatter or the beach must be compacted to a density above 60 percent of relative density. If an operating pond is available for detailed study, sampling and testing can be conducted from the dike inward for 800 to 1,000 feet toward the decant or slime area with a series of holes from the surface down to natural soil. From these tests the density and slope of the projected tailings pond can be planned prior to construction. A computer program for stability can then determine what must be done to attain a given height. Additional volume storage due to compaction is minimal, but effective compaction could raise the density above the critical density so that liquefaction would probably not take place in case of seismic shock. The low density of the material on the beach as it is deposited by spigots in the peripheral discharge method is the main problem with the upstream method of construction. For this reason this method is considered to be limited to low dams in nonseismic areas. A main advantage of this system is the low operating cost, compared with that of methods using cyclones on the pond area.

Another practice used in the upstream method that is detrimental to stability is the creation of a large borrow area by the dragline in building up the dike after each lift (fig. 29). When spigoting is continued from the top of the new dike, this borrow area is filled with total tailings before it overflows and runs to the water pool. This borrow area now has the run-of-mill tailings in it, which would have a lower permeability than the segregated sand when a beach is being formed. This places a barrier against the dike for the entire height of the dam, and while it is not continuous, it can be the source of a weak plane in the stability analysis (fig. 34). This condition is worse on a coarse grind with segregation on the beach because the water goes from a relatively permeable segregated sand and comes up against the less

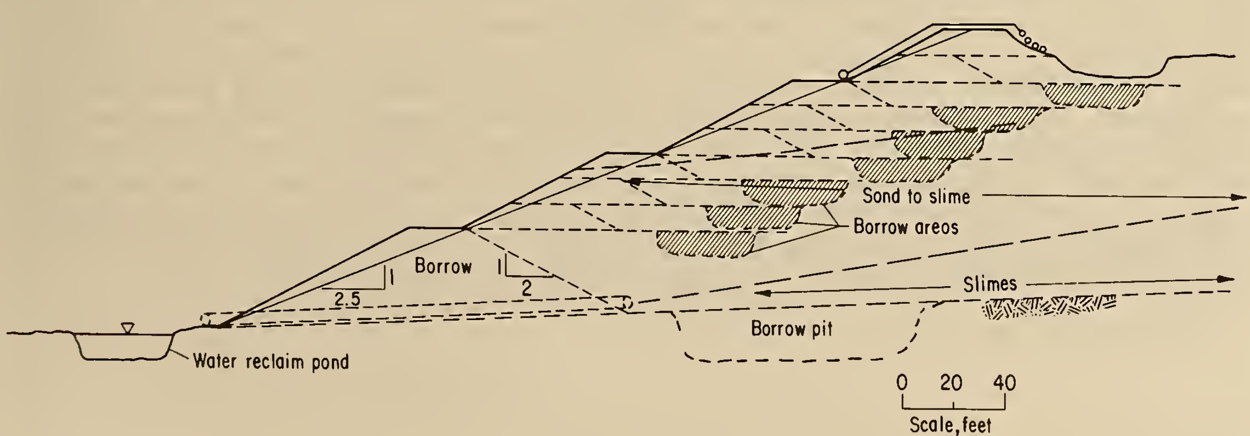


FIGURE 34. - Barrier formed by borrow area in upstream construction.

permeable material in the borrow area, which is contrary to the rule of having the permeability increase toward the outside of the dam. To remedy this situation, the borrow should be obtained from a wider area and pushed up to the dragline by a bulldozer to eliminate the deep borrow pit. In tailings pond with a fine grind and high pulp density, there is practically no segregation, so the material that fills up the borrow area is about the same as that which was excavated.

## Sand Separation

Figure 2 shows the screen analysis of some typical copper, iron, lead-zinc, gold, and molybdenum mill tails. These have a spread of 38 to 69 percent minus 200 mesh and represent the bulk of the metal mines in the United States. Figure 3 shows a typical curve of some phosphate slimes with 100 percent minus 20 micrometers and 26 percent minus 0.2 micrometer.

## Spigots

The most common method of placing tailings on a pond in upstream construction is by spigots spaced 16 to 60 feet apart along a header pipe which is installed on the embankment crest (fig. 28). The material immediately adjacent to the spigots is used for dike construction on the next lift and should be the best tailings material available. The gravitational separation of the sand and slime as it flows into the pond ranges from very poor to very good depending on the grind, specific gravity, and pulp density. Figure 9 shows the gradation of the mill tails from mine A (see below), the gradation at the dike, and the change in gradation 1,000 feet from the dike of a material that has about 38 percent minus 200 mesh and is spigoted at 30 percent pulp density by weight. Figure 10 shows the gradations for a material that has 58 percent minus 200 mesh and is spigoted at 48 to 50 percent pulp density. Figure 35 shows the uniformity of the permeability change in mine A; the vertical permeability is $8 \times 10^{-3}$ centimeters per second at the dike and decreases to $9 \times 10^{-4}$ centimeters per second 700 feet from the dike. The horizontal permeability had the same relative decrease in a 1,000-foot distance from the spigot point but was approximately five times the vertical. In mines B and C (fig. 36-37), the horizontal and vertical permeability in surface test pits is about the same at the dike as it is 500 feet from the dike and also near the decant area 3,000 feet from the dike. From very limited testing there is also no apparent difference in permeability between the horizontal and vertical samples from surface test pits. The vertical permeability does definitely decrease with depth and is very low just above the natural soil. These two examples might represent the extremes of what can be expected from spigoting. Mine B could certainly improve the separations of the sand and slimes if it were necessary by decreasing the pulp density 10 to 20 percent.

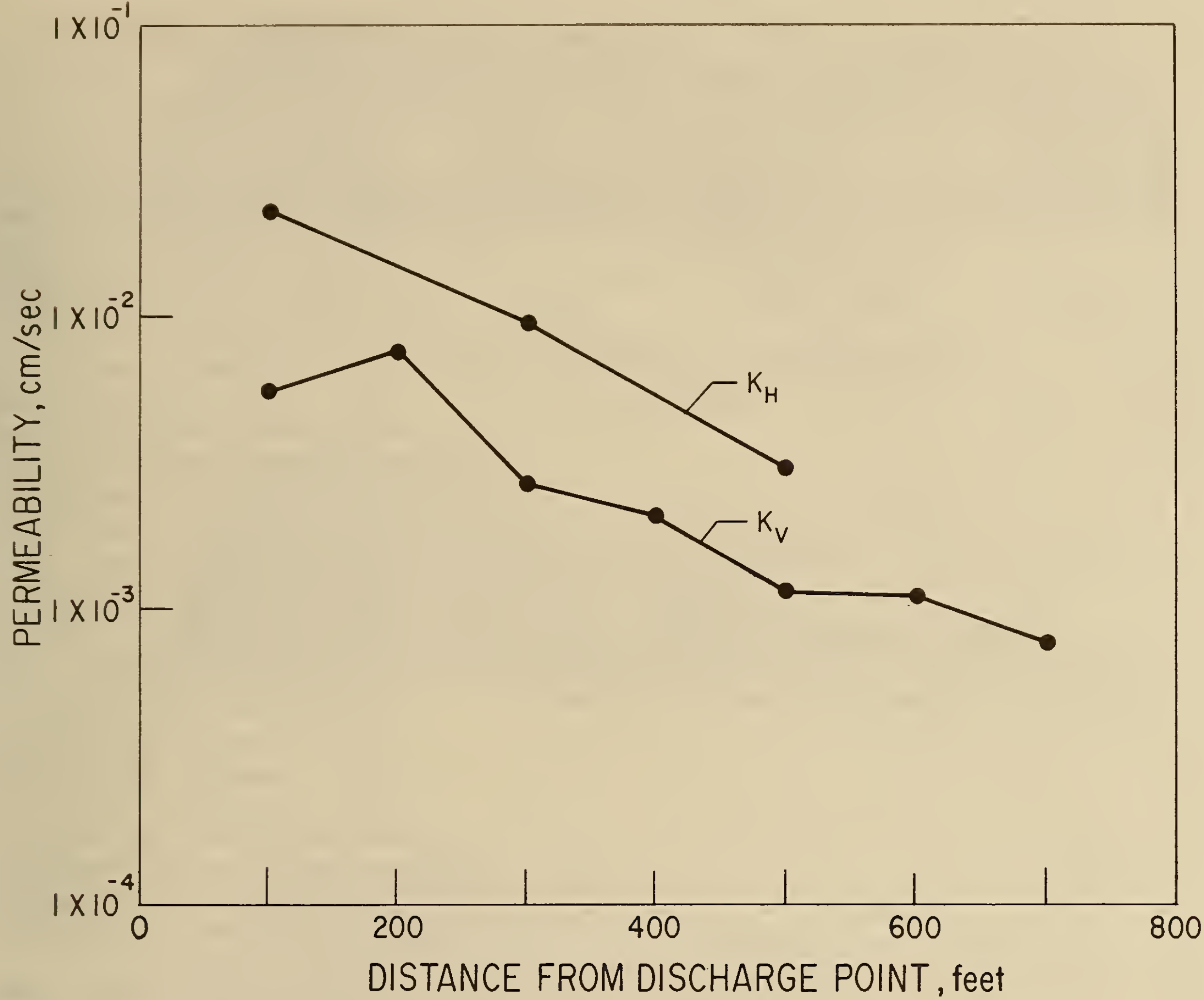


FIGURE 35. - Permeability, mine A—uniformity of change.

## Cyclones

As the grind in metal mines becomes finer, 50 to 60 percent minus 200 mesh, cyclones can recover a larger percentage of the sand for dike building than can spigoting, and they are being used for this purpose. They are usually mounted on the crest of the embankment but are sometimes mounted on an abutment at one side of the embankment. From the cyclones the fines are piped to the tailings pond, and the coarser sand drops directly onto the embankment or is repulped and discharged through spigots or launders onto the embankment. In some cases, the fines can be piped upstream many hundred of feet so that no slimes are beneath the dike area. In some cases the slimes may be piped to another tailings pond temporarily.

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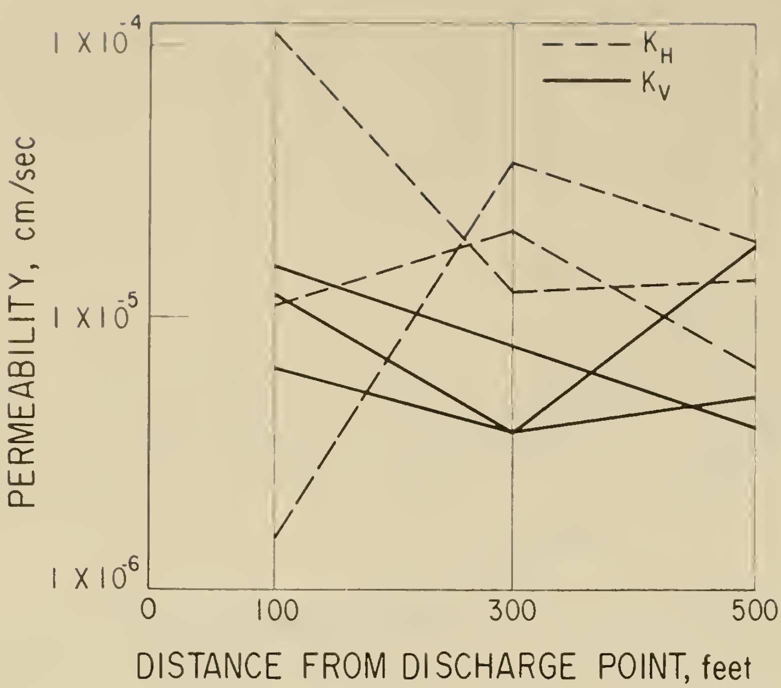


FIGURE 36. - Permeability, mine B—nonuniformity of change.

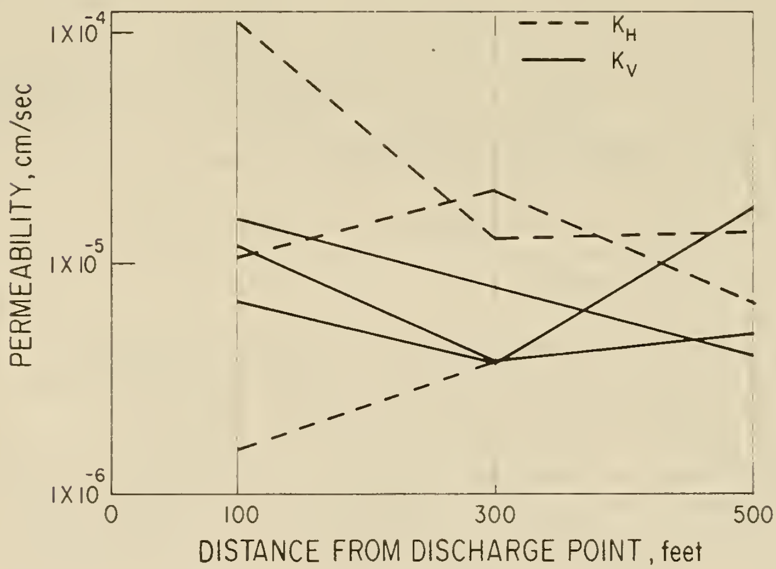


FIGURE 37. - Permeability, mine C—nonuniformity of change.

Pond Water Management

The movement (loss) of water in a tailings pond varies considerably between the start of filling and eventual abandonment. Initially, a pervious base has stability advantages because of water movement through the bottom, but as the height increases and the slimes accumulate over large areas, consolidation reduces the permeability of the base. As the height of the dam increases, the effect of the original base becomes negligible, and the tailings themselves have an overriding effect on the downward movement of water. A tailings dam that starts with a permeable gravel base will ultimately develop an almost impervious base. This is because the water can easily escape from the tailings into the natural ground, allowing the bottom layer to consolidate (fig. 38). Note that if the sands can be placed near the interior toe of the dike, the pervious base remains effective and will function like a longitudinal drain, and the lower phreatic line may be used for stability analysis. If one cannot assure this, the permeability of the entire base approaches zero, and the upper line must be used for stability analysis. As the structure increases in height, its condition could change from very stable to very unstable unless the outside is kept free-draining. Frozen conditions can also impair stability (fig. 39).

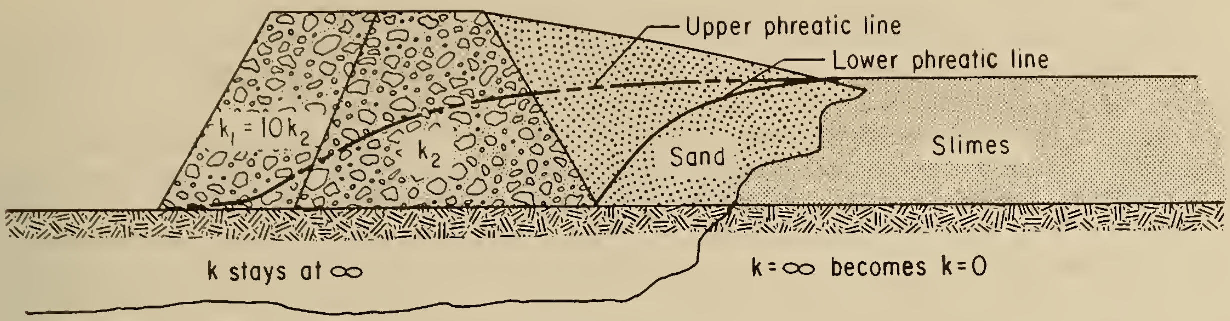


FIGURE 38. - Change of base permeability with time and material.

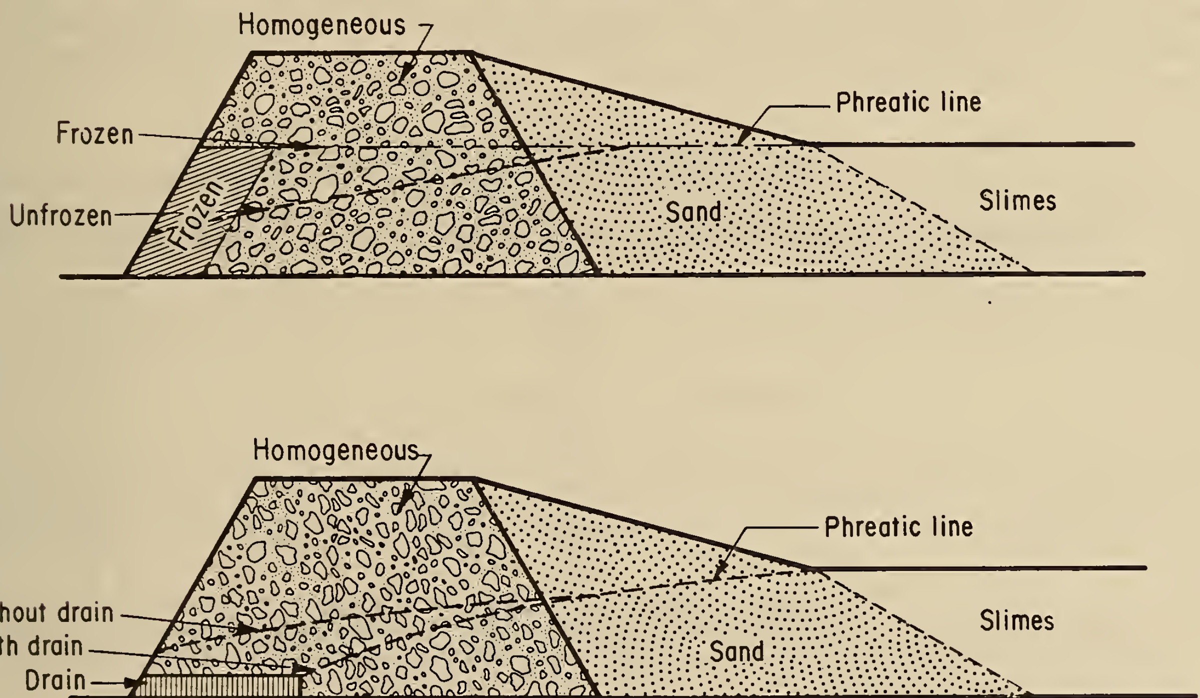


FIGURE 39. - Location of phreatic line for frozen versus unfrozen face.

The pool area may be as much as 25 percent of the total area of the impoundment, but it should be kept as small as possible to minimize evaporation losses in a hot, dry climate, or to restrict the size of the slime area and keep the water pool as far from the crest as possible. The evaporation losses in desert areas may be as much as 84 inches per year, which is an average of about 4 gal/min per acre of pond and 2 gal/min acre of wet sand. The decant system or barge pumps should be designed so that the water pool may be controlled within several inches.

## Embankment Freeboard and Wave Protection

Tailings embankments built of borrow material need protection from wave action if the water is allowed to come against the borrow dike. Wave action is a very destructive force and can erode and overtop an embankment unless ample freeboard is provided. The height of the waves depends on the wind velocity, the duration of the wind, the fetch (the distance the wind can act on the water), and the depth of water. On steep upstream slopes, riprap will limit the uprush of the waves to approximately 1.5 times the height of the waves and will prevent erosion by wave action. Tailings embankments should not have free water standing against the dike except where a water-type dam is impounding the tailings, and then they should have riprap on the upstream face. Table 6 gives the aproximate wave heights for various wind velocities and fetch, and the freeboard and riprap gradation for the 3:1 slope. For 2:1 slopes, the thickness should be 6 inches greater. A layer of filter gravel should be placed between the riprap and the embankment.

TABLE 6. - Embankment freeboard and wave protection

APPROXIMATE WAVE HEIGHTS

| Fetch, miles | Wind velocity, miles per hour | Wave height, feet |
|---|---|---|
| 1 | 50 | 2.7 |
| 1 | 75 | 3.0 |
| 2.5 | 50 | 3.2 |
| 2.5 | 75 | 3.6 |
| 2.5 | 100 | 3.9 |
| 5 | 50 | 3.7 |
| 5 | 75 | 4.3 |
| 5 | 100 | 4.8 |
| 10 | 50 | 4.5 |
| 10 | 75 | 5.4 |
| 10 | 100 | 6.1 |

FREEBOARD REQUIRED FOR WAVE ACTION

| Fetch, miles | Normal freeboard, feet | Minimum freeboard, feet |
|---|---|---|
| Less than 1 | 4 | 3 |
| 1 | 5 | 4 |
| 2.5 | 6 | 5 |
| 5 | 8 | 6 |
| 10 | 10 | 7 |

RIPRAP REQUIRED ON 3:1 SLOPES FOR PROTECTION AGAINST WAVES

| Reservoir fetch, miles | Nominal thickness, inches | Gradation, percentage of stones of various weights (pounds) | | | | |
|---|---|---|---|---|---|---|
| | | Maximum size | 25 percent greater than-- | 45 to 75 percent | | 25 percent less than[1]-- |
| | | | | From | To | |
| 1 and less | 18 | 1,000 | 300 | 10 | 300 | 10 |
| 2.5 | 24 | 1,500 | 600 | 30 | 600 | 30 |
| 5 | 30 | 2,500 | 1,000 | 50 | 1,000 | 50 |
| 10 | 36 | 5,000 | 2,000 | 100 | 2,000 | 100 |

[1]Sand and rock dust less than 5 percent.

Source: U.S. Bureau of Reclamation.

Wave action is not a problem if even a small beach is provided between the dike and the water, because the waves dissipate harmlessly in the shallow water on the beach.

The crest width should be no less than 12 feet for easy equipment operation in a situation where water is against the embankment. The most suitable crest width will depend on the allowable percolation distance through the embankment at full pond level, the height of the structure, and ease of construction. In the upstream, downstream, and centerline methods using tailings to raise the embankment, this does not apply because the water is not in contact with the dike. The width is then governed by the equipment used.

The approximate crest width for embankments under 100 feet high is given by the question:

$$W = \frac{z}{5} + 10, \tag{4}$$

where $W$ = crest width in feet,

and $z$ = height of the crest above the foundation at its lowest point.

Tailings dams over 100 feet in height should have crests not less than 30 feet in width.

## CORES, BLANKETS, AND MEMBRANES

With water quality becoming ever more important, tailings dam design must incorporate methods that limit seepage into the groundwater and downstream to the greatest extent economically possible. Water conservation by recycling also must be a part of the design.

Where the only economically available materials for starter dam construction make a pervious embankment, the seepage water that comes through the starter dam or through drains into the downstream area must be collected and either recycled or given necessary treatment before it is released to the drainage. Seepage, as such, is no problem if it is controlled. If seepage must be reduced or if the entire tailings area is on deep pervious soil which prevents collection, then impervious cores, blankets, or membranes can be used to reduce the seepage from the pond.

Seepage can be controlled when using pervious construction materials by placing a vertical or inclined zone of impervious material at the center or upstream from the center of the dam. When a relatively impervious layer is at a shallow depth below the foundation, a core trench can be cut through the foundation into the impervious layer to extend the core. A core trench is illustrated schematically in figure 40.

In conjunction with an impervious core, a blanket or membrane may be used to extend the impervious zone upstream of the embankment. This increases the head loss and the length of the seepage path through the pervious foundation.

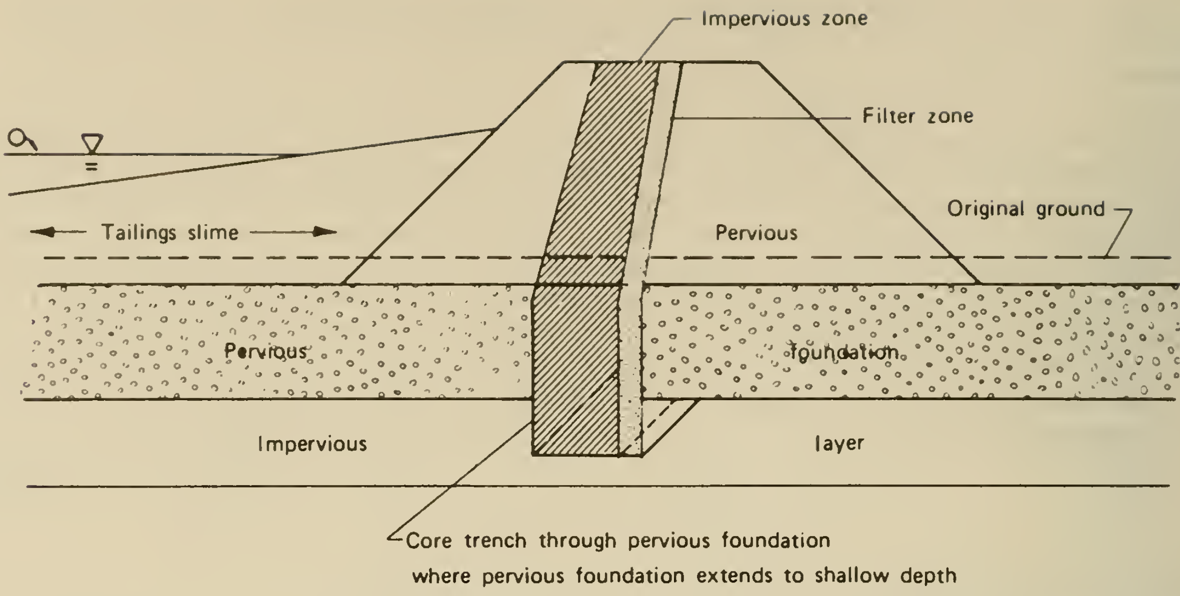


FIGURE 40. - Core trench.

It decreases the seepage losses from the pond and decreases the uplift from the hydrostatic pressures beneath the dam. The amount of seepage that is tolerable will determine the necessary extent and thickness of the upstream blanket or membrane to accomplish this. The extent of most tailings ponds is so great that plastic films or liners are too costly. Clay blankets and cores have been the usual method of reducing seepage. These have low permeabilities but are not readily available in some areas. The area of the impervious blanket should be large enough that the estimated seepage is not above the allowable maximum when the high range of permeability of the blanket material is applied (fig. 41).

In specific situations such as in areas of shallow bedrock, it is best to catch all seepage for recycling or treatment and release (fig. 42).

## Sealing the Bottom of Tailings Ponds

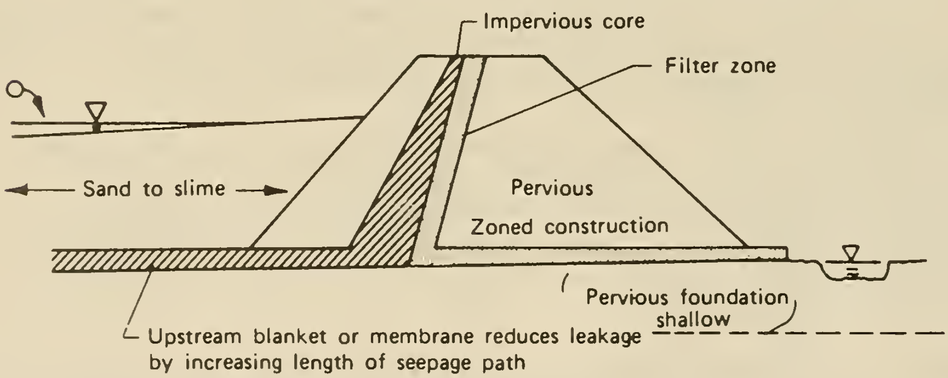


FIGURE 41. - Impervious zones—cores.

Tailings ponds in areas with deep soil (more than 50 feet to bedrock) of high permeability may be required to reduce seepage into the ground water because of contamination from chemicals, metallic ions, dissolved solids, or

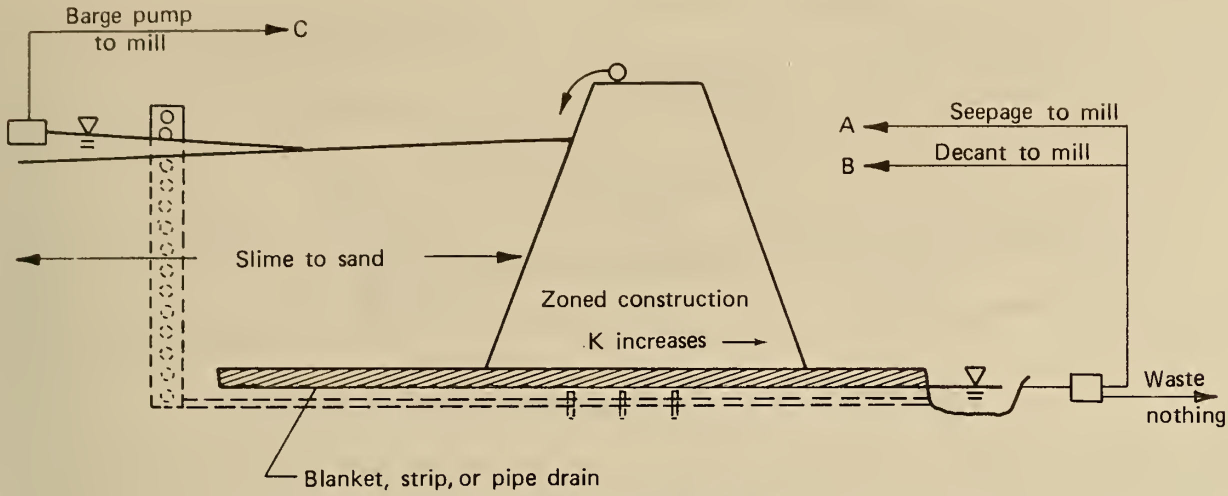


FIGURE 42. - Closed system.

other deleterious materials. The usual tailings pond has a clear water-soil contact on the upstream side which can account for 75 to 95 percent of the seepage from a pond (fig. 43). In relatively flat terrain this zone can be several hundred feet wide and with a permeability of $10^{-2}$ to $10^{-4}$ centimeters per second can transmit a tremendous amount of water into the subsurface.

Because of the large areas used, tailings ponds may be 5,000+ acres in size, and the usual sealing materials such as asphalt, rubber, plastic, etc., are prohibitively expensive. The tailings themselves can be used to cut down this flow by placing a layer of slimes on the natural soil first. If the grind is fine enough (50 to 60 percent minus 200 mesh) the entire tailings flow can be used to make a seal about 5 feet thick. The flatter the area, the easier it is to do this. If the tailings have a smaller percentage of minus 200-mesh material, it may be necessary to cyclone the product to get material

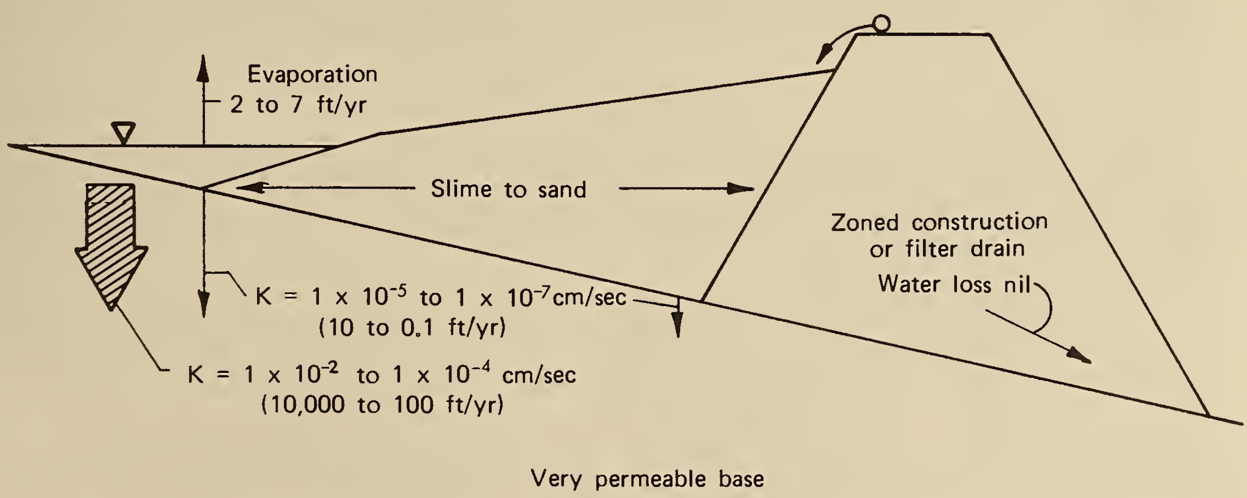


FIGURE 43. - Clear water-soil contact.

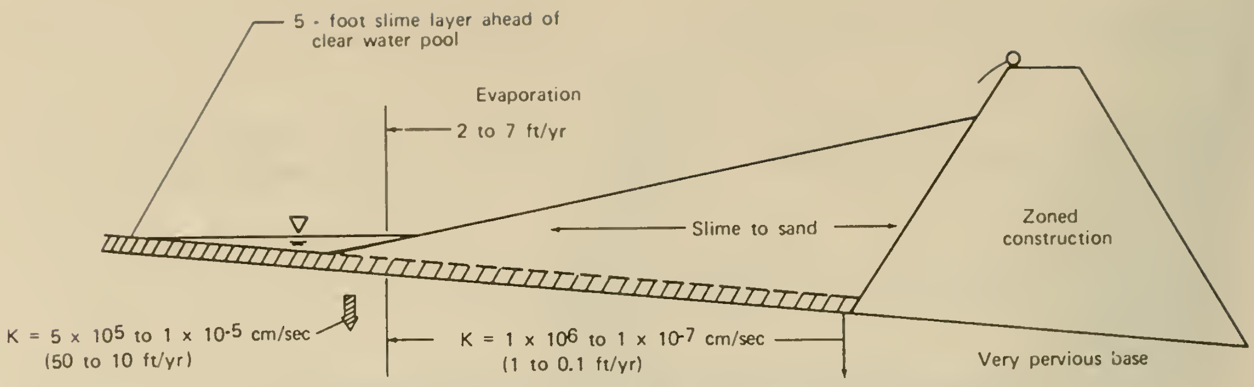


FIGURE 44. - Five-foot slime layer at bottom of pond.

fine enough to make a temporary blanket with a permeability of $5 \times 10^{-5}$ to $1 \times 10^{-5}$ centimeters per second or slower (fig. 44). This is much more impervious than the $10^{-2}$ to $10^{-3}$ centimeter-per-second permeability of many soils. When the temporary seal is in place, normal spigoting or cycloning can be initiated, and the operation can proceed in the regular manner. Thus, the seepage velocity into the ground water is reduced from 1,000 to 10,000 feet per year to 10 to 50 feet per year. As the elevation of the pond increases, the depth of the slime increases, and with a permeable base and consolidation, the permeability through the bottom of the pond may be reduced to $1 \times 10^{-6}$ centimeters per second, 1 foot or less per year.

The method used to place a layer of slime over the entire surface of a tailings area will depend on the terrain to be covered, the grind, and pulp density. Temporary dikes can be used to separate the area into small pools, or long low dikes parallel to the contours, making long narrow pools, may be used. Spreading the slime through sprinkler pipes and close control of spigoting from all four sides of a tailings area are also possible methods. Whatever method is used, there will be some seepage into the natural soil initially, but this will be minimal and for a very short time as compared to seepage with no seal at all. When using the upstream method, drains should be placed at the upstream toe of the starter dike for water control.

## Hydraulic Barriers

Where the tailings dam is constructed on a thick pervious foundation, and pollution control requirements preclude the escape of water from the tailings pond, seepage losses may be controlled by developing a hydraulic barrier downstream of the tailings dam.

The hydraulic barrier (illustrated on fig. 45) can be produced by a line of pumping wells and a line of injection wells downstream of the embankment, the injection wells being located downstream of the pumping wells. Fresh water is supplied to the injection wells, while ground water is extracted from the pumping wells. If the piezometric water levels along the line of the injection wells are maintained at elevations higher than the piezometric water levels along the line of the pumping wells, a hydraulic barrier will be formed

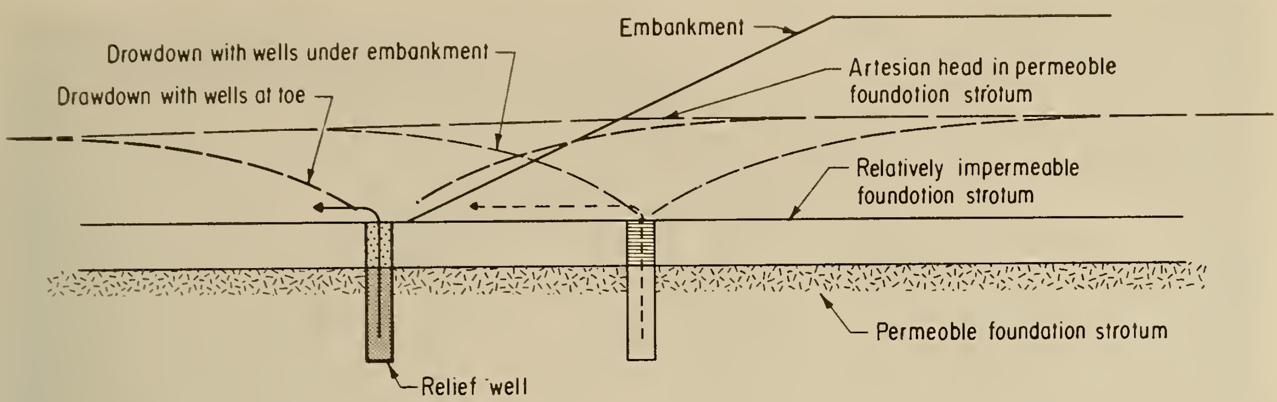


FIGURE 45. - Pumping wells for hydraulic barrier and relief wells.

that will prevent the flow of seepage from the tailings pond past the line of pumping wells. This should be checked in the field by piezometric measurements. The hydraulic barrier may be usable for up to 100 feet of overburden, but would not be feasible where the depth was much greater. This method does not eliminate contamination of the ground water in the long run, but only as long as the injection and pumping wells are operating.

## INSTRUMENTATION

All tailings embankments that are to be more than 100 feet high should be monitored during construction and operation and for some time after being abandoned. Many factors can affect the stability, and instrumentation should be installed in the embankment and foundation to monitor these changes. Instruments may be installed to measure piezometric levels (water), seepage flows, embankment movement, and total pressure.

### Piezometers

A simple and effective piezometer is the Casagrande type (fig. 46), which is a porous hollow stone. It is installed in a hole drilled into the embankment or foundation with a 1/2-inch pipe to the surface. The water level is measured by an electric probe lowered down the hole. In large-diameter holes several piezometers can be installed in one hole separated by bentonite plugs and surrounded by filter material. This type of piezometer requires a relatively large amount of water to register a change in height because the 1/2-inch tube must be filled. The response, especially in low-permeability material, is much slower than that of the pneumatic piezometer, which measures pressure only. Under certain conditions less sensitive types can be installed using perforated steel casing or well points with 1-1/2- or 2-inch pipe.

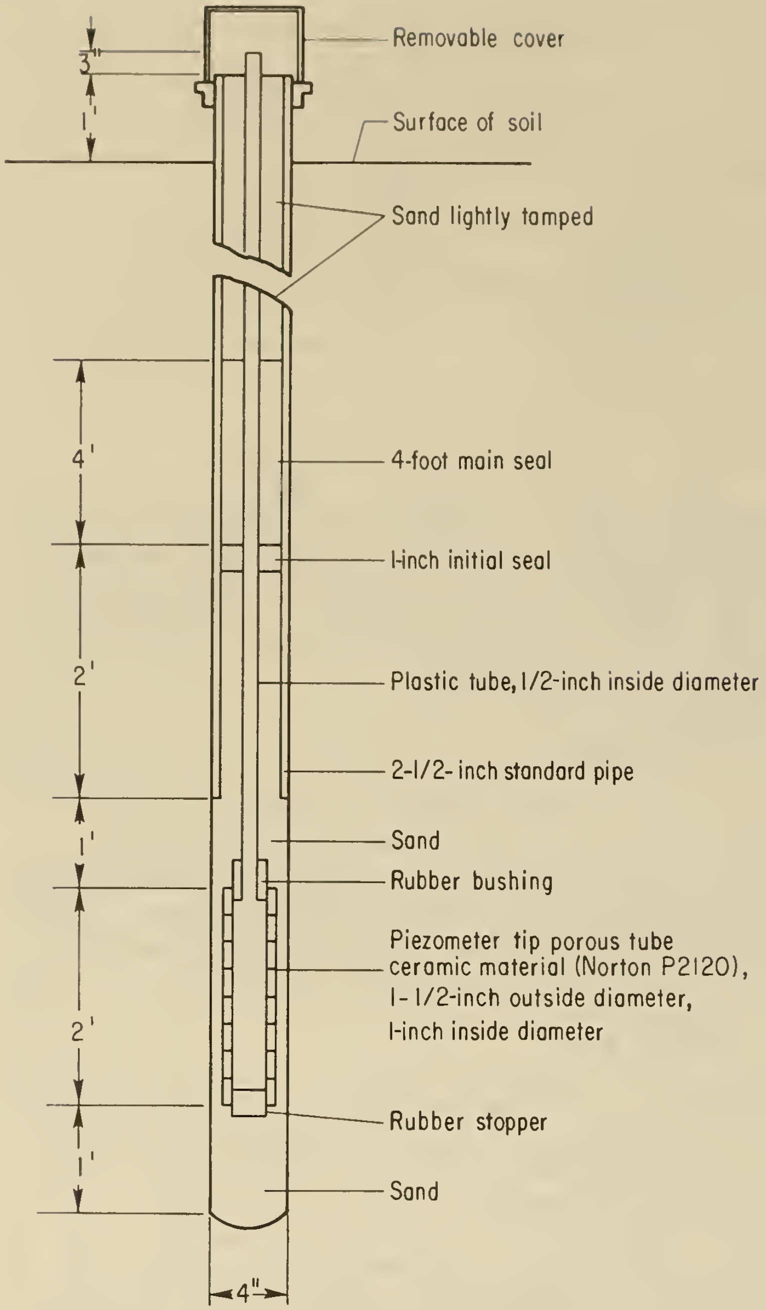


FIGURE 46. - Porous-tube piezometer installation (Casagrande type).

The pneumatic piezometer (fig. 47) has been developed to a high degree of accuracy and is very sensitive to immediate changes in pore water pressures; there is no need for actual flow of water--only the water pressure change. These piezometers can be placed on the ground surrounded by protective filters prior to construction or filling of the embankment with the lead lines extended out to the top of the starter dam so they can be monitored for the full life of the embankment. Additional piezometers can be placed on the beach or in the slime zone as the embankment rises to check for perched water tables or excess pore water pressures. These should also be protected by filters when they are installed. They can be installed in old embankments by drilling holes to the depth desired, perforating the casing at the elevation the piezometer is to be installed, and surrounding it with filter material and a bentonite plug above and below. In some cases the casing can be pulled for salvage after the piezometer is installed, or the piezometer can be installed through hollow-stem augers without casing. Readings should be made of all piezometers on a regular schedule, and records kept.

If the dam is to be built of material for which no waterflow information is available, the phreatic surface can be estimated by assuming typical parameters and using the LaPlace equations for flow-net solutions or by the finite-element method. Computer programs have been written for the finite-element method to predict the location of the phreatic surface (43). The program used


FIGURE 47. - Pneumatic piezometer and readout.

in this study utilizes the finite-element method to determine pressures and flows as governed by Darcy's law. Correct hydrostatic head, geometry, and horizontal and vertical permeability according to the inclination and layering are the only input data required for predicting the phreatic surface. Using these values, the computer will search through the numerous phreatic surface lines and adjust pressures at the finite-element nodes so that equipotential lines and the phreatic surface can be obtained in a matter of minutes.

Typical flow lines that can be developed are illustrated in figures 48-50. Note the effect of embankment stratification and the corresponding horizontal permeability ($K_h$) and vertical permeability ($K_v$) on the location of the flow line and on the required width of longitudinal drain (fig. 48), the relationship between downstream shell permeability relative to foundation permeability and the saturation level in the shell (fig. 49), and the effect of zoning and stratification on the location of the saturation line (fig. 50).

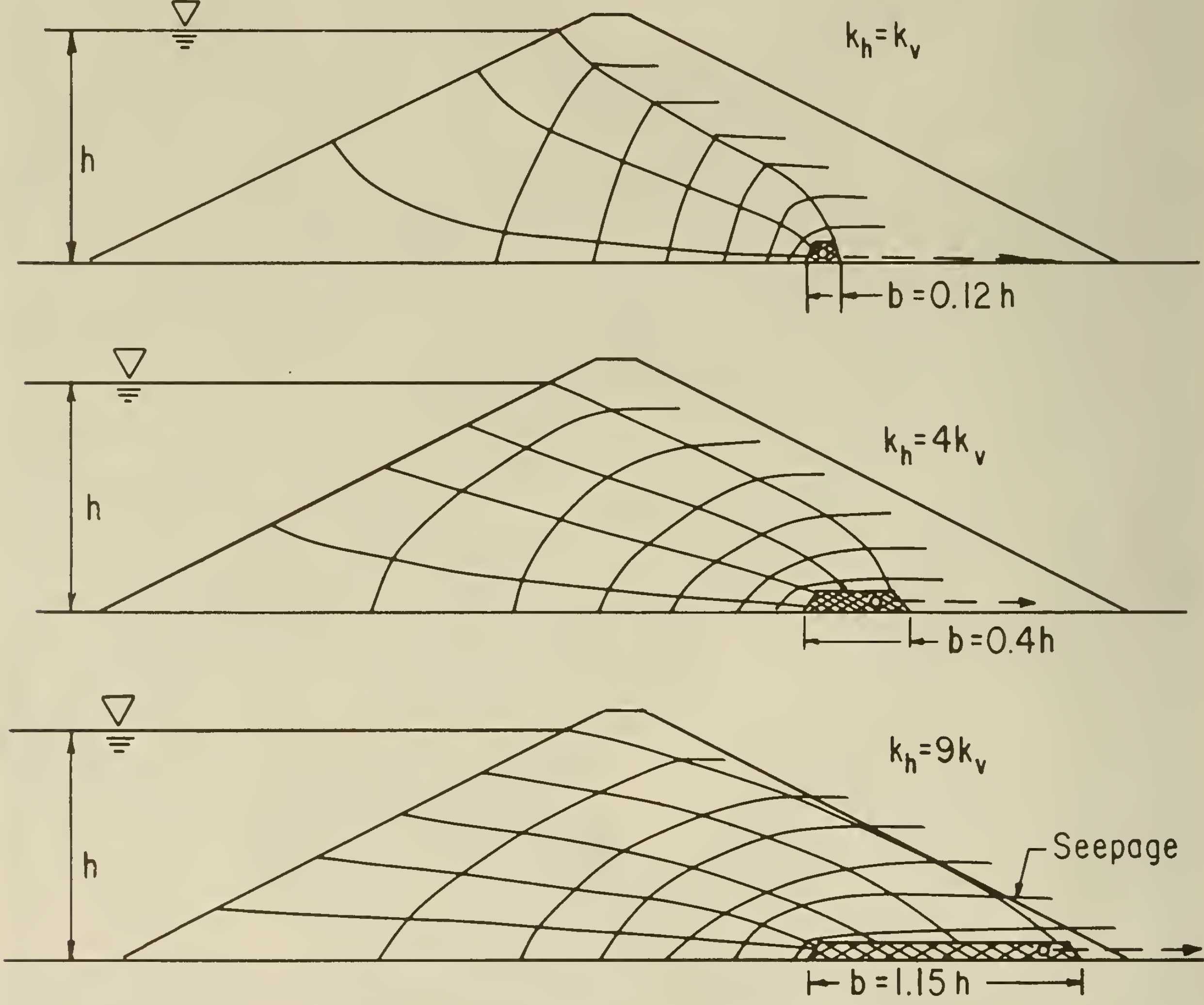


FIGURE 48. - Effect of embankment stratification on required width of longitudinal drains in homogeneous dams (19).

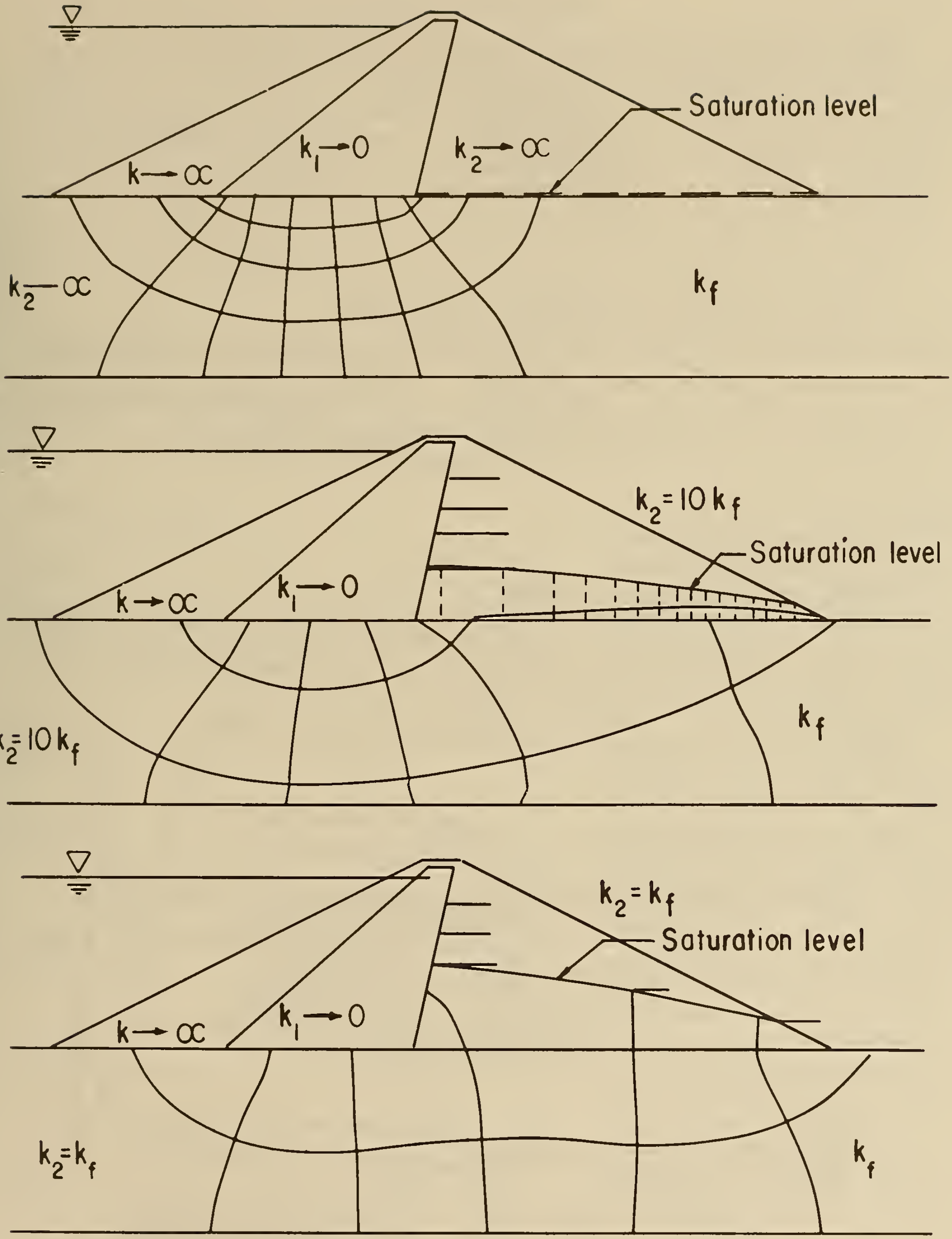


FIGURE 49. - Zoned dam study (19).

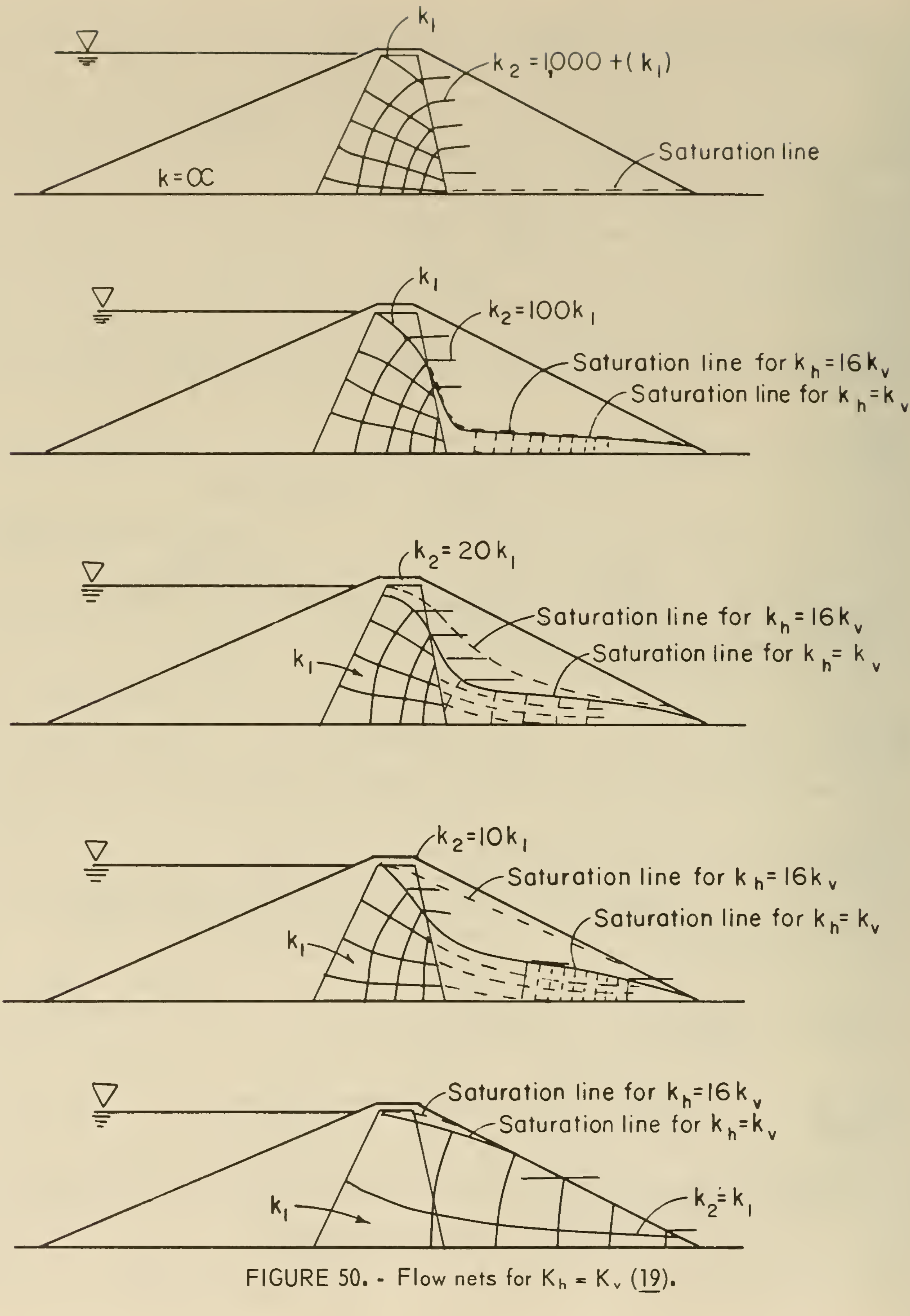


FIGURE 50. - Flow nets for $K_h = K_v$ (19).

## Movement Indicators

One of the simplest but most effective ways to measure the movement of an embankment is to drive pipe or rebar vertically into the berm of an embankment in a straight line of sight so that any movement can be measured by simply measuring the offset from the line of sight. On cross-valley dams the permanent stations can be placed on the natural ground or rock off the embankment with the recording stations set on an abandoned berm where they can be protected from the spigoting or dike building operation. These should be measured for elevation changes also. In flat country the survey line must be brought up to the embankment from the natural ground at each end of the downstream slope, and the movement markers placed the same as described for the cross-valley dams. This is important because the line of site must be made from points on top of the sand berm, and these points themselves could have considerable movement which then would not show the true movement of the other points. The permanent points on the natural ground at each end should be placed at a considerable distance from the embankment so as to be unaffected by the foundation deformation.

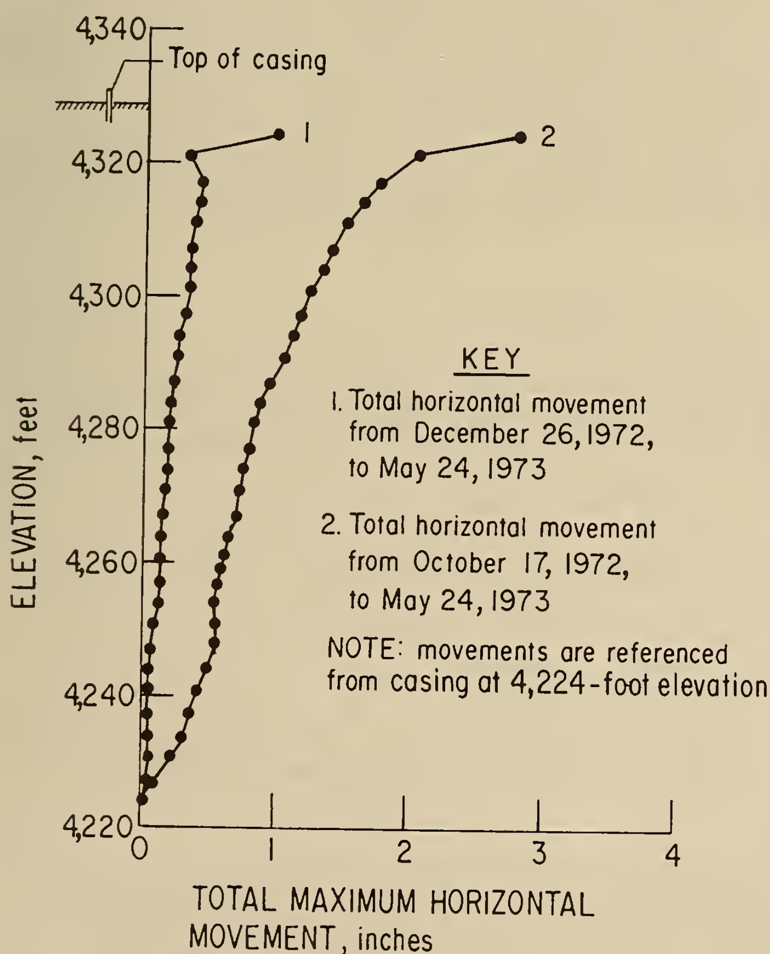


FIGURE 51. - Slope indicator data.

Another device for measuring both horizontal and vertical movement is the slope indicator, which is lowered into a casing with grooves which guide it and measures the movement in two directions at right angles. For this instrument the casing must be set into bedrock, or if the alluvium is deep, the bottom of the casing must be set into the foundation and the casing installed in the embankment as it rises. Another way to install the casing is to drill a hole and install the casing after an embankment is constructed.

From the readings of the slope indicator a profile of the hole can be drawn showing where the movement is taking place from top to bottom (fig. 51).

## Pressure Cells

Pressure cells should be placed on the top of the

decant lines or other culverts that will be under the embankment to measure the pressure of the soil and water against the plane surface. The reinforced-concrete conduit should be designed for the combined weight of material at maximum height, but the pressure cells placed at strategic spots along the line could verify the actual pressures. This information can be valuable in designing future conduits.

### Records

All instruments installed in an embankment should be read and recorded on a regular basis, which at startup should be quite frequently. Later, as patterns are established, the time between readings can be altered to suit the situation. These records are very important and should be reviewed periodically to see if additional records are needed and to note the trends. The piezometric level measurements are probably the most important and have a direct tie-in with the seepage or flow from the drains. Precipitation measurements should be made, and positions of the spigots or cyclones that are operating should be noted. Often the length of time a group of spigots can be left in operation depends on the rise of the piezometric level in that area. A rise in the piezometric level could also mean that the drains were beginning to plug up. Records should be kept of any changes in construction and waste placement procedures, and of changes in grind or pulp density to the pond. These data should all be presented in graphical form so that variations and trends can be readily noticed. These records should be reviewed constantly by the person responsible for compiling the records, and periodically by the designer of the embankment, who should be well qualified to interpret the results.

## SLOPE STABILITY

Since no slope can be regarded as permanently stable, slope stability is a relative matter. However, in soils engineering practice, the term is used in reference to the possibility of a sudden relatively deep-seated slide.

Soil and rock materials fail in shear if the applied shearing stresses on any surface exceed the shear strength of the materials along that surface. The resisting forces are the shear strength of the materials, both frictional and cohesive. The cohesive strength is minimal or negligible in most cases. Pore water pressure at the failure surface lowers the resistance to sliding because it reduces the effective stress. Artesian water from the substrata into the embankment will also reduce the stability of the embankment owing to hydraulic uplift. The shear strength of the materials can be further reduced by weathering and softening by water. The shear strength may be increased by compaction or by chemical cementing of the waste materials.

Cracking of the embankment caused by differential settlement can reduce the shearing resistance along potential failure surfaces. This cracking may lead to slide failures or piping. Dense tills are usually strong, their shear strengths can have both frictional and cohesive components, and they may be relatively impermeable and incompressible. Drilling and sampling are necessary to seek out inconsistencies in the materials.

Sands and gravels are relatively incompressible, and their shear strength is primarily frictional with no cohesion. Here again, the density, gradation, and particle shape determine their behavior. Loose, fine sand acts the same as the same gradation material in mine tailings. If it is saturated and below the critical density, it is subject to liquefaction under shock load. Silts develop strength from either friction or cohesion, depending on density, gradation, and moisture content.

Clay in the foundation may cause embankment settlement and instability. As the embankment rises, the clay may consolidate and gain shear strength. Uncompacted clays in waste piles saturate and swell, reducing their shear strength to almost zero. The finer portions of tailing from metal mines act as clays. The entire output of some mines is mudstone or clay and requires specially designed dams to contain it. Phosphate slimes are also a special case because of the fineness of the clay and the pulp density of the material being impounded.

## Safety Factor

The index of stability with respect to a sudden failure is known as the factor of safety (FS) of the slope. In the most general terms, this may be defined as the ratio of the potential resisting forces to the forces tending to cause movement. The factor of safety may also be defined as that factor by which the shear strength parameters must be divided ($C'/FS$ and $\phi'/FS$) to bring the potential sliding mass into a state of limiting equilibrium. The stress-strain characteristics of most soils are such that relatively large plastic strains may occur as the applied shearing stresses approach the shear strength of the material. Thus, a slope which is on the verge of failure has a factor of safety of 1.0, but in the design the FS must be greater than 1.0 so that the strains will not exceed tolerable limits, and to allow for differences between the pore water pressures and the shear strength parameters assumed in the design and those that actually exist within the slope. The factor of safety of tailings embankments has not yet been determined by law but will very likely be set at 1.5 or greater. Each embankment must be considered on its own merit, based on the following criteria:

1. Location relative to population centers.

2. Total mass (volume and length of life).

3. Rainfall in the area and drainage into the embankment.

4. Mineralogy.

5. Type of terrain.

In other words, a tailings pond within a city or beside a main road or railroad should have a static factor of safety of 1.5, while one in a remote area many miles from civilization would be considered very safe with 1.3. The dynamic factor of safety is generally less, as indicated by a modified Bishop

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method using a $\phi$ of 20° and C of zero with pseudostatic gravity forces of 0.1 and 0.2 gravity (51).

The following shows the effect of seismic loading and a high phreatic surface.

Static 1.67 FS
At 0.1 gravity 1.23 FS
At 0.2 gravity 0.95 FS (failure)

The reason for the 1.5 factor of safety is that, by its very nature, sampling of a tailings pond and testing these samples in the laboratory is somewhat difficult. Other considerations are that the embankment is being built higher each year, and the geometry and pore water pressures change with the seasons and elevation. The analysis must be made with reference to the worst conditions which may exist or may develop. If the physical properties testing is not of the weakest material within the embankment or foundation, the FS of the analysis will not be the lowest and therefore will not be a true assessment of the stability.

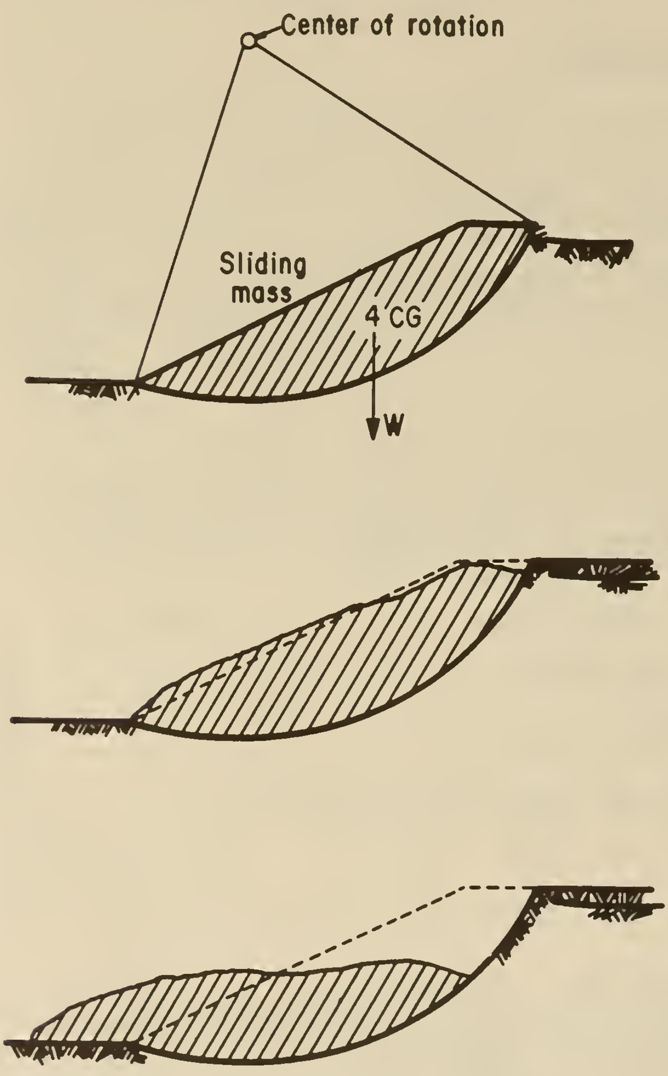



FIGURE 52. - Plan of rotational slide.

## Rotational Slides

A somewhat idealized concept of a rotational slide is shown in figure 52. The crosshatched areas of the figure represent a cross section of the sliding mass. The surface on which sliding occurs is curved and may often be approximated in cross section by a circular arc. The sliding tendency is created by the moment of the mass about the center of the arc. This moment is opposed by shearing resistance developed along the sliding surface. When all available resistance is overcome, failure occurs as shown in the bottom panel of figure 52. Two pictures of a typical rotational slide are shown in figure 53.

These findings were reported by the Swedish Geotechnical Commission after an investigation of a number of slides along the Swedish railways. As a result, the

FIGURE 53. - Rotational slide.

circular arc failure is known as the Swedish circle method of analysis. Hand calculations for the factor of safety of a given embankment by the "method of slices," utilizing trial and error, is a long and tedious process and with the advent of the computer has practically been eliminated. Many good programs have been developed for the computer to calculate and select the failure circle.

## Computer Analysis

Numerous programs have been written utilizing the Fellenius method, the original method of slices, and the Bishop method. The Fellenius method develops a conservative estimate since it completely neglects the side forces on the individual element slices. The Bishop method is the more accurate. A study of all of these programs has resulted in our selection of the modified Bishop program, originally written at the Massachusetts Institute of Technology and slightly modified by the Bureau of Reclamation to include both the Bishop and Fellenius factors of safety (14). The Bureau agrees with R. V. Whitman and W. A. Bailey that this is the best program available today (52).

The factor of safety is defined as the moments about the center 0 for the circular failure arc ABCD, as shown in figure 54, and is described by the equation

$$FS = \Sigma \frac{(\bar{C}\ b\ \sec\ \delta + \bar{N}\ \tan\ \phi)}{\Sigma\ W\ \sin\ \delta}, \tag{5}$$

where $FS$ = safety factor,

$\bar{C}$ = cohesion for soil, lb/sq ft,

$b$ = width of slice,

$\delta$ = element angle,

$\bar{N}$ = effective normal force,

$\phi$ = friction angle,

and $W$ = cohesion for soil, lb/sq ft.

The denominator of the equation for FS is an expression for the moment of the weight of the soil in the failure mass. (The radius r cancels since it occurs in both the numerator and the denominator.) The numerator is the moment of the shear stress about 0 along the failure surface.

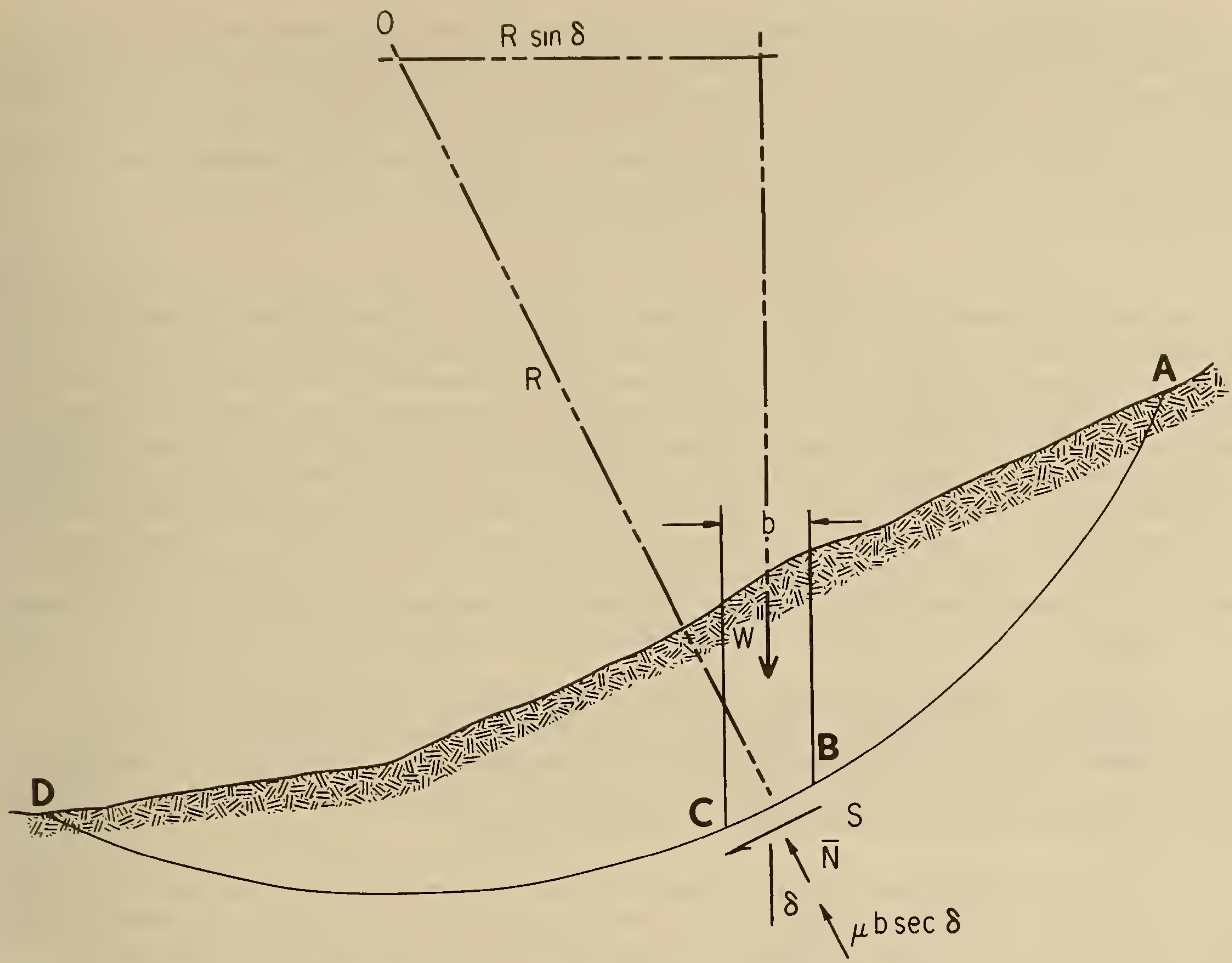



FIGURE 54. - Circular failure arc.

$\overline{N}$ is determined for the simplified Bishop method by the sum of forces in a vertical direction, according to the equation

$$\overline{N} = \frac{W - b \sec \sigma \left(\mu \cos \delta + \dfrac{\overline{C}}{FS} \sin \delta\right)}{\cos \delta + \dfrac{\tan \phi \sin \delta}{FS}}, \tag{6}$$

where $\mu$ = pore pressure, lb/sq ft.

Therefore,
$$FS = \frac{\Sigma\left[\overline{C}b + (W - \mu b)\tan \phi\right] \dfrac{\sec \delta w}{1 + \dfrac{\tan \phi \tan \delta}{FS}}}{\Sigma W \sin \delta}. \tag{7}$$

Since FS appears on both sides of the equation, it must be solved by successive approximations. This is the equation for which the complete program was written.

The computer will systematically search through the numerous failure circles as outlined in the guidelines of the program. It will select a circle with minimum FS (the plane along which failure will normally occur) while also computing the individual circles so that the effects of the geometry, etc., can be studied in each case. At the end of the program the computer will list the FS of all the circles selected for that particular embankment and conditions.

The simplest way to illustrate the mechanics of the computer program is to present an example. Figure 55 shows the soil properties and location of minimum failure circles for two different conditions that were analyzed. Case 1 assumes a high phreatic line, and case 2 assumes the water table at ground level. Although the geometry and soil structures are identical for both analyses, the difference in the location of the phreatic line does alter some of the soil properties, as shown in figure 55. By comparing the computed minimum safety factors (0.997 versus 2.287), one can see that the location of the phreatic line is very critical for determining stability. A factor of safety of less than 1 is a failure situation, above 1 is a stable situation, and 1.5 is considered a safe design value, particularly if the soils are somewhat coarse and not likely to be subjected to seismic activity.

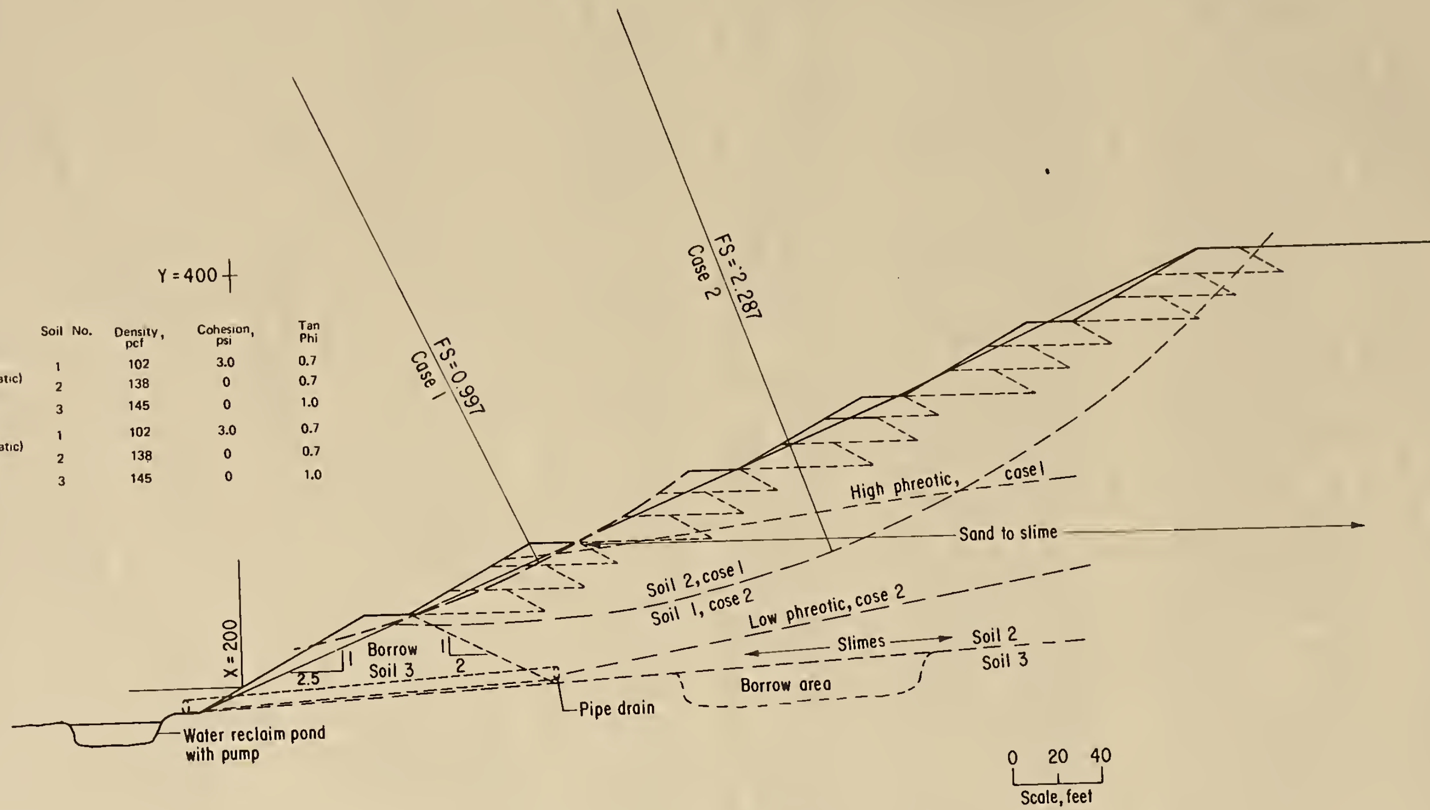
Complete computer listings of input and output for cases 1 and 2 have been included as appendix D for those who would like to study the computer data searching and analyzing procedures in more detail. Not all of the failure circles tried are listed, but the factor of safety is computed for both the Fellenius and Bishop methods for those listed.

Should the embankment conditions indicate other than a circular arc failure, a second computer program using the Morgenstern and Price procedure can be used (37). With this program any desired geometric failure plane can be described and the factor of safety attained in a few minutes of computer time. Unlike the Bishop program, it does not search for the minimum safety factor but produces only the input failure plane. Since the development of computers, the most difficult part of the slope stability analysis is providing correct input data. This point cannot be overstressed.

## Computer Input

In most cases it is extremely difficult, if not impossible, to obtain soil samples that are truly representative of the zone being studied. Consequently, the soil properties developed from these samples must be interpreted and applied with great care. Assuming that input values developed are representative of the actual case being studied, the computer factors of safety are general guidelines and are meaningful only if used in conjunction with all of the other design considerations.

The engineer must anticipate and design for the worst possible situation; that is, ultimate height, maximum phreatic line, saturated soils, and seismic activity. The safety factor will only be meaningful if such values are used for the computer program.




| | Soil No. | Density, pcf | Cohesion, psi | Tan Phi |
|---|---|---|---|---|
| Case 1 (High phreatic) | 1 | 102 | 3.0 | 0.7 |
| | 2 | 138 | 0 | 0.7 |
| | 3 | 145 | 0 | 1.0 |
| Case 2 (Low phreatic) | 1 | 102 | 3.0 | 0.7 |
| | 2 | 138 | 0 | 0.7 |
| | 3 | 145 | 0 | 1.0 |

FIGURE 55. - Sample of computer analysis.

Grain-size distribution, the area of the tailings pond, and the rate of discharge have much to do with the stability of the embankment. A finer grind in the mill with the coarse fraction being taken out for use as underground fill will make dike building more difficult. Combine these factors with the small impoundment area compared with the tons of waste per day, and the situation becomes worse. The tailings used underground at some properties do reduce the total that must be impounded on the surface by 40 percent or more, but they also remove the coarse sand (the best material for building the dike) and the coarse sand beach, which provides added safety.

An example of rapid building is a situation in which a 500-ton-per-day operation is impounded in a 5-acre site with a pond rise of 1 foot in 33 days. Even a 10-acre site with a 1-foot rise in 66 days is much too fast. Such situations could cause a rapid increase of pore pressure because the water does not have time to percolate through the fine material. Even with the best of conditions, a rapid building rate is not good, and every effort should be made to keep the annual rise compatible with the seepage ability of the soil or drains.

Piezometers installed in proper places in the embankment will allow monitoring of the pore-water pressure in the dam, which can be related directly to the safety factor as shown in figure 56, a typical graph showing the variance of safety factor with phreatic water height in the dam. This type of chart

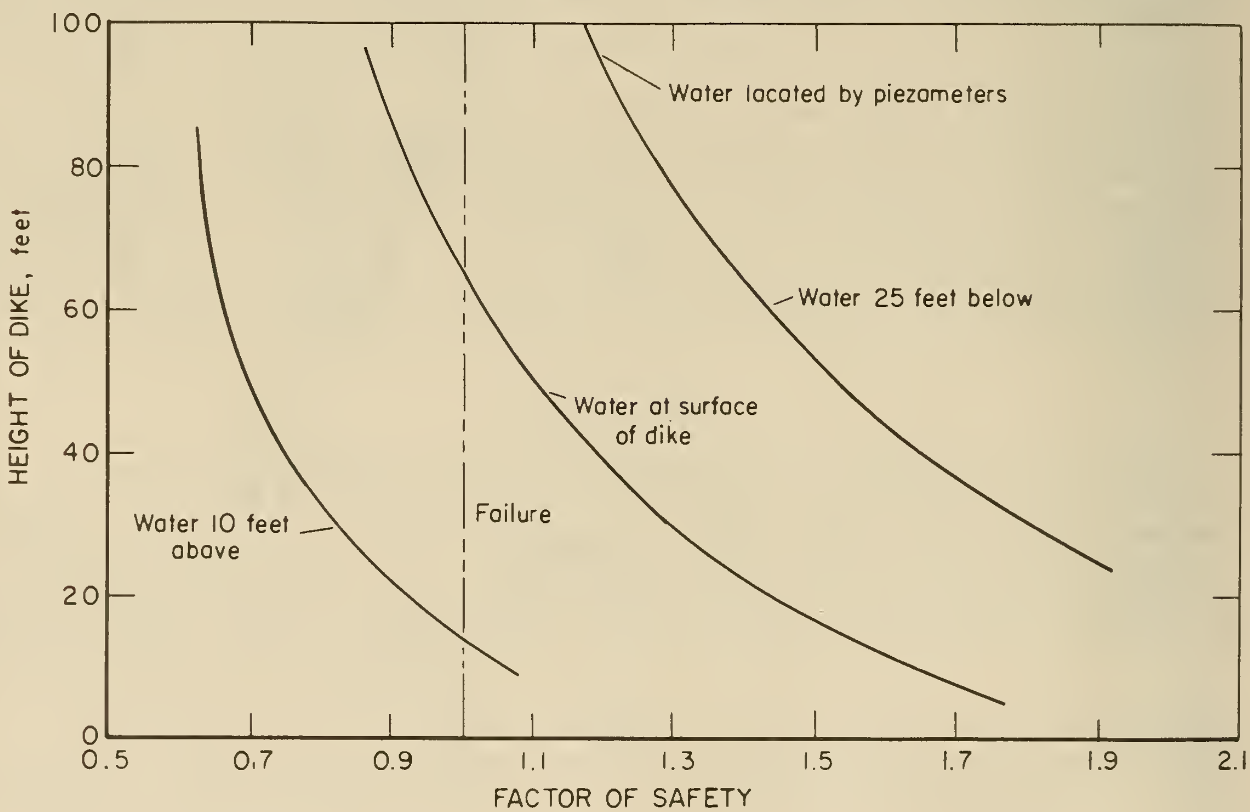



FIGURE 56. - Factor of safety from phreatic water height in dam.

can be developed for any dam and used by the operators to predict the safety of the embankments. The phreatic surface is related to the rate that material is placed around the periphery of the dike.

Safety factors cannot be accepted at face value if there is the possibility of liquefaction from either earthquake, sonic blasts, or sudden load. Soils in the 80- to 280-mesh sizes, saturated and above the critical void ratio, are very sensitive to liquefaction. Soils finer than that would be too sluggish in their reaction to shock because of lower permeability, and those coarser would dissipate the water fast enough to make failure from shock unlikely.

All soil testing requires stringent testing conditions and experienced personnel. Since Coulomb's theory and stability equations are approximations, these samples have to be studied thoroughly by experienced soils engineers to insure that true values are obtained for any stability analysis (32). Once these values have been interpreted, it is simple to predict the stability using the computer. Ultimate height, slope, and water movement can be determined by soil engineering analyses. Before any major construction of this type, these analyses should be made either by the operating firm or by a consulting agency. The cost is only a few percent of the total investment.

## MAINTENANCE AND INSPECTION

The active life of mine tailings embankments may be from a few years to as much as 100 years; during this time many changes can take place that affect the stability of the embankment. This type of construction is radically different from a water-type dam where the construction is done in a relatively short time under close quality-control of material and methods.

The physical properties of the tailings used in pond construction may change over the years for many reasons. Such changes can alter the stability of an embankment, resulting in variations in the factor of safety. One of the most common changes is the increase in tonnage to the mill without a compensating change in tailings area; this will mean an increase in the annual rise of the dam, reducing the factor of safety. A change in grind with an increase in the minus 200-mesh material can cause a higher phreatic line, a possible decrease in the efficiency of the drains, and increased seepage through the starter dam. Any one or combination of these changes can mean a decrease in the factor of safety of the embankment. It is therefore important that a continuous program of inspection and maintenance of the embankment be started at the beginning and maintained throughout the life and even after the abandonment of the embankment. The records of the instrumentation as described previously are one of the most important aids in determining the safety of the tailings dam, and in a high dam are an absolute necessity. High embankments should be thoroughly inspected by a competent engineer at least twice a year during the active life of the pond. A review of the records of the instrumentation in the embankment should be included in this inspection.

Daily inspection should be made of the spigots or cyclones, the decant lines, and position of the water pool in relation to the decant or the

tailings area boundary. The drain lines should be checked for quantity of water and sediment.

The objectives of the quarterly or semiannual inspection and maintenance program should be to determine--

1. Whether there are any major changes in the foundation or the embankment that were not anticipated in the design such as heaving of the foundation at the toe, longitudinal or transverse cracks in the crest, or excessive seepage.

2. Whether the material characteristics have changed; and if so, how will these changes affect the stability?

3. Whether the distribution of the material into the pond is as the design called for; and if not, how will it affect stability?

4. Whether the slime and water pond is where it should be in relation to the dike area (gradually moving upstream in the cross-valley ponds and within close bounds around the decant towers in the flat-country ponds).

5. Whether the dam construction is rapid enough to keep the slimes and water well back from the dam.

6. Whether the decant towers or barge pumps can handle the storm runoff in addition to the reclaim water.

7. Whether the drains are operating, free of sediment, and flowing at a regular rate.

8. Whether there is seepage on the downstream face of the starter dam indicated by weeds growing along the face, or worse yet, seepage on the downstream face of the sand dam above the starter dam.

9. Whether the decant lines are intact and free of cracks that could allow sand to pipe into the lines and cause a total failure. These can be visually inspected where the decant lines are large enough.

10. If the phreatic surface is as planned; or is there excess pore water pressure from within the foundation or perched water tables?

11. Whether there have been variations in the water levels or a sudden rise in the water level, the appearance of any new springs, or new seepage on the face of the embankments, foundation, or abutments. Conditions at the seepage exit points, decant and drain pipe outlets should be reviewed: is the water clear or does it contain sediments; is there sloughing in the area; is water coming along the outside of these pipes; are there sinkholes in the beach or slime zone which would indicate piping which should be seen in the drain or decant water?

12. Whether there has been an increase in embankment movement as indicated by the surface control points or slope indicator.

13. Any evidence of borrow from the toe or any other area of the embankments that might affect stability.

14. Whether the embankment geometry is no steeper than planned.

15. Whether diversion channels and pipes have withstood spring runoff or storms. Are they adequate and in good repair?

All of these points must be watched closely to check stability, but if a tailings dam has been designed for a total height of 500 feet and all seems to be going well at a height of 200 feet, a thorough investigation should be made by drilling holes into the fill material. Undisturbed samples can be checked for inplace density, screen analysis, $\phi$ angle, and cohesion. With this information and the phreatic surface and geometry of the embankment, a static and dynamic stability analysis can be run to get the FS at that time. From this information the FS can be projected for an embankment 500 feet high to determine if the slope can be steeper or must be flatter, or if the construction must be stopped short of the design height.

If an older mine has had successful tailings disposal for a number of years using a certain procedure for a given nominal height and is making plans for a new dam, testing should be done as outlined above on the old tailings area prior to making plans for the new ones. In this way if anything has to be done differently, it could be started at the design stage. Some of the things that might have to be changed are improved drains or drain protection, flatter overall slope on the downstream face, compaction on the dike, and compaction of the beach for a distance of 500 to 700 feet to increase the density and shear strength.

These items would have to be started at the beginning of construction to be effective. Compaction of the beach might be something new that would be desirable if a high dam was to be built.

## Seepage Control in an Operating Pond

If a tailings pond has been designed, constructed, and operated in the best manner known at the time, and still has seepage that is not controlled or is excessive, there are remedial measures that can be employed.

Seepage through a tailings dam is to be expected, but the design should be such that it is controlled. The phreatic surface or line of saturation is the surface at which the pressure in the seepage water is atmospheric, and it is analogous to a ground water table. This line is also the upper boundary flow line of a flow net, and it is very important to keep this line of saturation well below the downstream surface of the dam. If this line is too high, the seepage water will break out on the slope. If the velocity of the water below the breakout point is great enough, erosion and sloughing of the soil may start, leading to piping and eventual failure of the dam.

Seepage, as such, is not harmful unless the amount is excessive or the water is of such poor quality that it degrades the drainage or ground water. Drains can be designed and built to catch the seepage water and return it to the pond or to the mill.

If the downstream face of a tailings dam has excessive seepage, it is caused by one or more of the following:

1. Improper design.

2. Improper or poor construction and supervision.

3. Failure of, or insufficient, drains.

4. Too rapid annual rise.

5. Spigoting too long in one area.

This can be corrected by reducing the downstream slope (fig. 57) or by placing a surcharge (fig. 58) of material on top of the affected area. Coarse mine waste used as a surcharge would need a protective filter blanket between the dam and the coarse rock to prevent piping. A clay or clay-alluvium surcharge would also need a drainage blanket on the face of the dam to prevent a buildup of pore water pressure which would raise the phreatic line, further compounding the problem. Cycloned sand could be placed directly over the affected area, provided its permeability is greater than that of the dam material. Any action taken must lower the phreatic line and increase the factor of safety.

Decreasing the pond area may reduce seepage if the seepage has spread over the full length of the dam. If the seepage is localized, it could be the result of leaving the spigots in one place too long, or it could be artesian water or springs from inside the pond area that were not noticed during site exploration or construction.

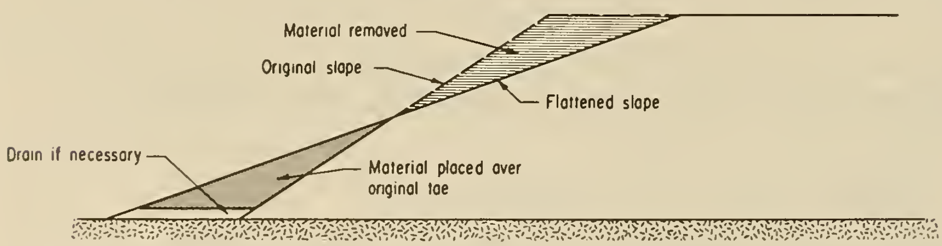



FIGURE 57. - Reducing downstream slopes.

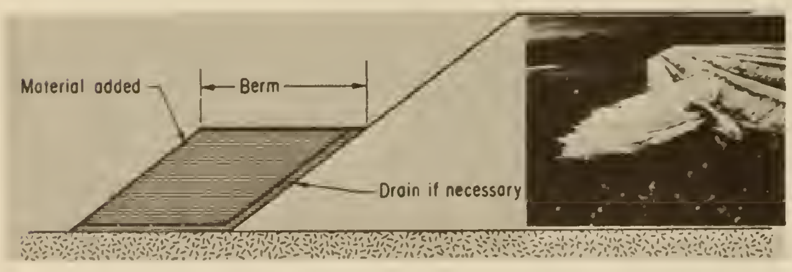



FIGURE 58. - Surcharge on toe of slope.

Seepage along pipes through the starter dam is very difficult to stop so the pipe must be designed with seep rings and properly compacted when placed.

Broken or crushed decant lines are also nearly impossible to repair unless they are large enough for men to enter and work in them. There is no substitute for proper design and construction with a good factor of safety to insure that there will be no rupture due to differential settlement on poor foundations. Drains and decant lines should be monitored for sand discharge, which would indicate seepage from the tailings area. Sinkholes in the top of the pond area are an indication of piping.

## Surface Drainage Control

Erosion on the downstream face of tailings piles can be controlled by grading the berms to slope upstream with drop pipes leading down to the natural soil at the downstream toe every few hundred yards along the periphery of the embankment. This limits the amount of runoff down the slope to only that amount which falls on the slope and does not accumulate and cause excessive erosion on the lower slope (fig. 59). These berms must be kept graded, and drop pipes and screens must be inspected and kept clean and free of weeds and other debris, especially after heavy rains. If the surface erosion can be controlled and the soil is not too acidic, grass or other natural vegetation can be grown on the slopes. This would require extra care, fertilizer, and water. Coarse rock or gravel can also be used to stabilize the slopes and help prevent excessive erosion.

Wave action can be a problem where the water is allowed to come in direct contact with a water-type dam or with a borrow dike. Even a small beach formed by spigot or cyclone discharge prevents damage by allowing the energy of the waves to be dissipated into shallow water on the beach.

Unnecessary surface drainage into a tailings area should be eliminated by diversion ditches or culverts, especially if the drainage area is large. The control of the maximum potential drainage should be included in the design of decants or storage volume. These diversion ditches and culverts should be very carefully designed, constructed, and maintained so that they are in good working order when needed.

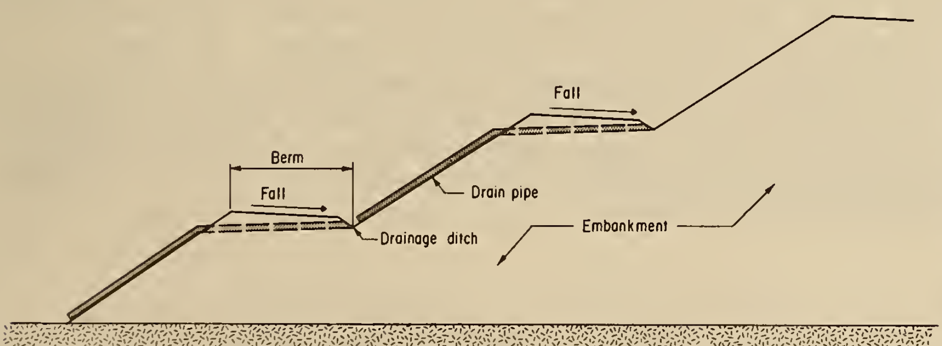



FIGURE 59. - Berm and slope drainage.

When tailings embankments are completed and abandoned, it may be necessary to maintain the decant system, or a permanent spillway may be constructed to prevent the acculation of rain water or surface

runoff. Each property will have different criteria governing what is needed. A tailings dam should never be breached as a means of disposing of water from the area, as erosion would eventually put all the material into the drainage.

## Remedial Measures

Some remedial measures required to maintain or improve the stability of an embankment may be minor in nature, while others could be a major undertaking. For some errors in design there is no remedy especially after the tailings pond is 40 to 50 feet high. Some methods of improving stability and reducing erosion follow:

1. By removing material from the crest of the slope and placing it on the toe, the driving force tending to produce a slide is reduced and the resisting force is increased, which increases the factor of safety. When material is placed on the toe, it must be more permeable than the material on the toe so that it does not impede drainage. A drain may be necessary if the material has a lower permeability than that on the toe (fig. 57).

2. Reducing the downstream slope can be done in another way by surchargin the toe area with waste rock or borrow material. Once more, care must be taken to insure proper drainage. If the area to be surcharged has seepage coming out the face or even a remote possibility that seepage might start with increased height, a filter blanket must be placed on the face and foundation base before the surcharge material is placed. This blanket is necessary especially where the material is to be relatively less pervious, but it is also needed where a coarse rock surcharge is to be used, to prevent piping into the coarse rock. This remedial work increases the resisting forces against failure and reduces the overall slope on the downstream face (fig. 58).

3. Berms are used in the design and construction of a tailings embankment to attain a given overall slope on the downstream face. The geometry of the embankment is important in the overall stability and should be determined during the design stage so as to contribute to the factor of safety. Berms are the common practice and serve other purposes as well. They serve as an access roadway and a place to lay the tailings pipe on the dam. They step back upstream about 30 feet for each 30-foot rise in elevation. As the embankment rises, the berm width can be increased if necessary for stability. The overall slope of the face of the embankment is determined by the shear strength, which varies with the size, shape, and mineralogy of the particles, as well as density and pore water pressure. Other remedial methods are height reduction and inverted filters (fig. 60).

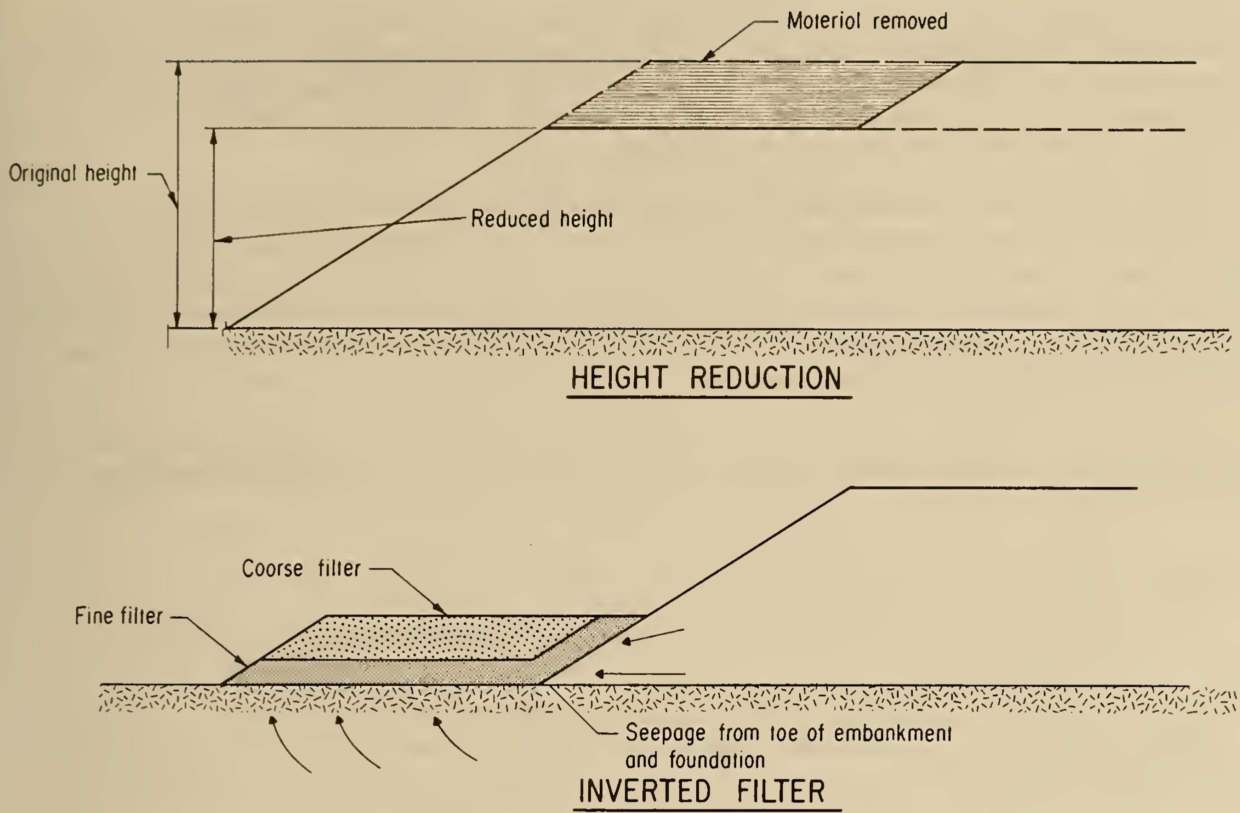



FIGURE 60. - Remedial methods.

## SUMMARY

Tailings ponds can no longer be allowed to "just grow." Because of the tremendously large daily tonnage, high land costs, and the limited availability of disposal sites, a properly designed embankment is necessary to keep the costs as low as possible and still be safe.

Unless the company engineers have acquired considerable expertise in the design of tailings embankments, the services of a competent and experienced consultant should be used. These services should be obtained for the design, periodic inspections (semiannual or annual), and stability analyses. Good construction and daily operation procedures have been developed for many mines and need not be changed unless they are the cause of unstable conditions or are inefficient.

Tailings disposal methods, the characteristics of the waste, the topography of the storage area, and the climatic conditions are so variable that it is impossible to make a design guide to cover all situations. Each tailings embankment should be designed to suit the conditions as they exist. The utilization of methods and suggestions included in this report should aid in decisionmaking.

The importance of sufficient area for the daily tonnage, adequate drainage to keep the phreatic line low, good operating procedures, and instrumentation to monitor the pore water pressure and the embankment movement cannot be overemphasized. The startup of a large-tonnage operation is especially vulnerable to problems related to sufficient tailings storage.

No attempt has been made to deal specifically with Federal, State and local permits, regulations, and restrictions in the construction and operation of tailings ponds with regard to air and water pollutions. Reducing seepage into ground water and recycling all tailings water have been emphasized. Dust abatement can be accomplished by proper operating procedures or by spraying the surface with an adhesive.

It is the authors' hope that this publication will aid those responsible for the proper design, construction, and operation of tailings embankments.

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## APPENDIX A.--TAILINGS POND EVAPORATION

To arrive at a water balance for the tailings pond, the evaporation rate must be known for the particular area. This can be reliably estimated by using data on solar radiation, air temperature, dew point, and wind movement. This information can also be obtained from daily measurements of evaporation from Class A pans. The lake evaporation rate ranges from 0.6 to 0.8 of the pan rate; an average figure of 0.7 is generally used. This procedure is described in detail by Harbeck, Kohler, and Kobug (25)[1] and will not be detailed here. For most calculations the pan coefficient and pan and lake evaporation can be taken directly off figures A-1 through A-3. In desert areas lake evaporation can be as high as 86 inches per year, which is more than 4 gallons per minute per acre. The wet areas of a pond not covered by water may lose as much as half this figure. This loss is difficult to measure but may be quite small as compared to the total evaporation loss, depending on the area and the fineness of the grind.

[1]Underlined numbers in parentheses refer to items in the bibliography.

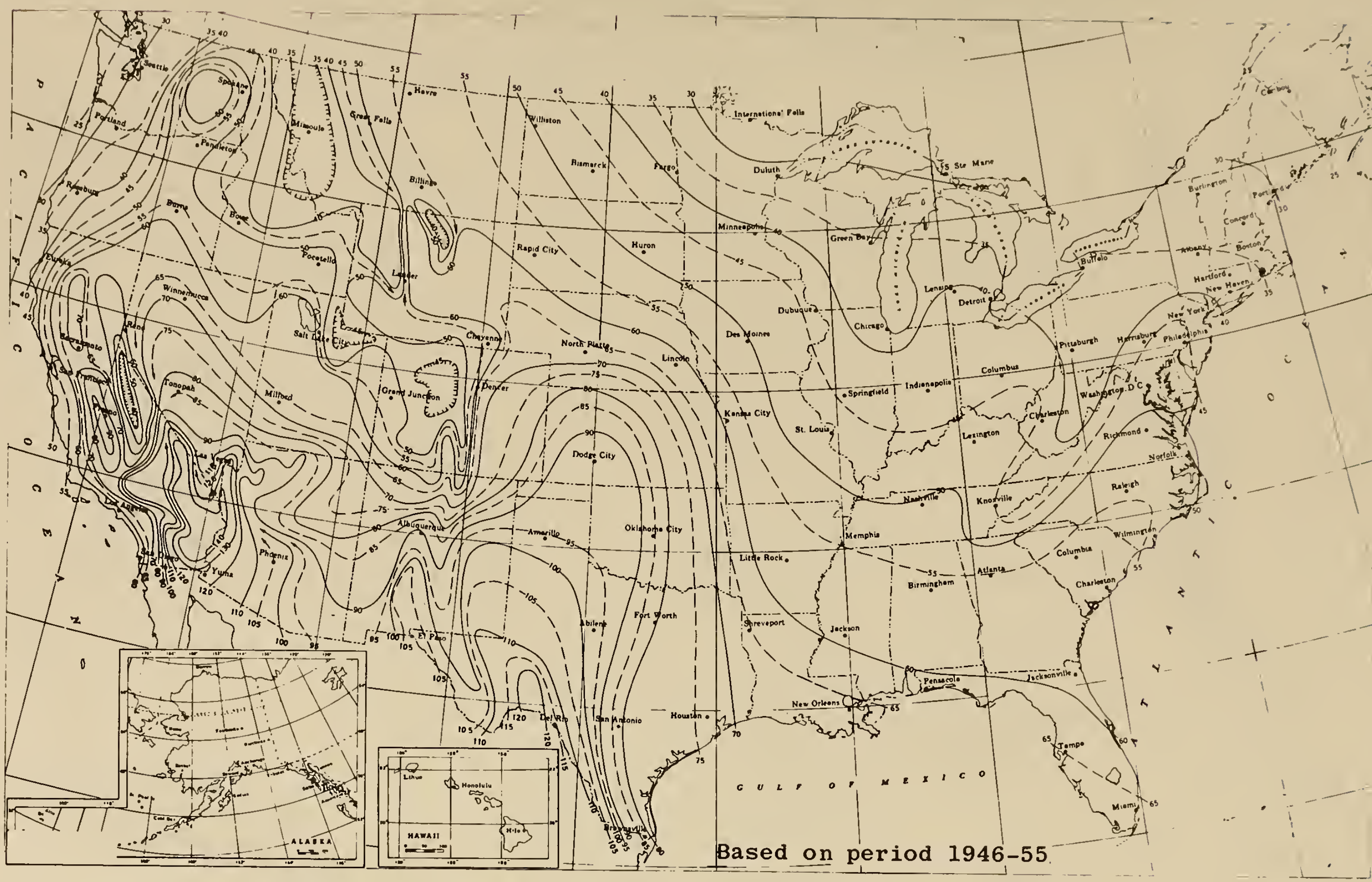



FIGURE A-1. - Mean annual class A pan evaporation, in inches (25).

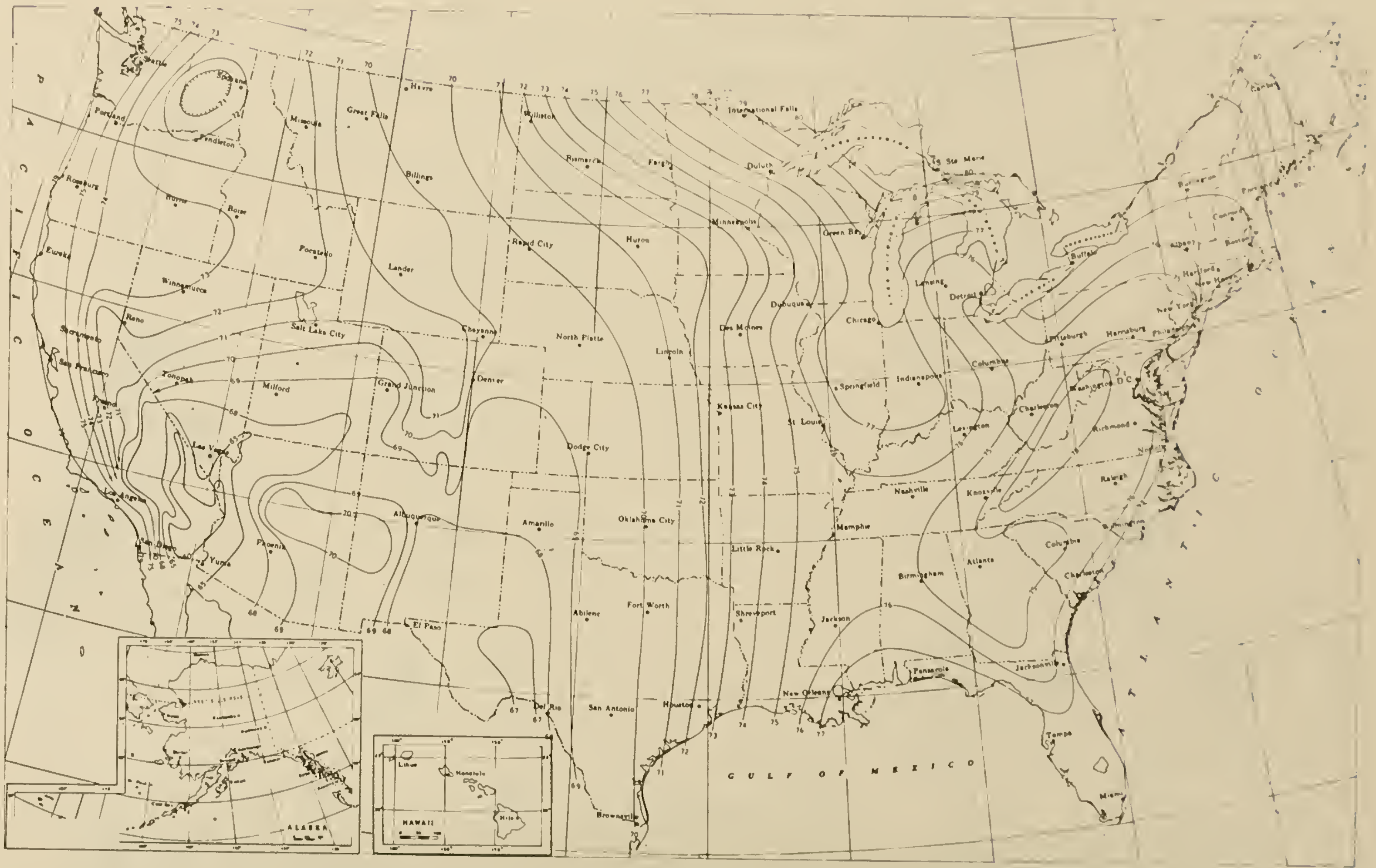



FIGURE A-2. - Mean annual class A pan coefficient, in percent (25).

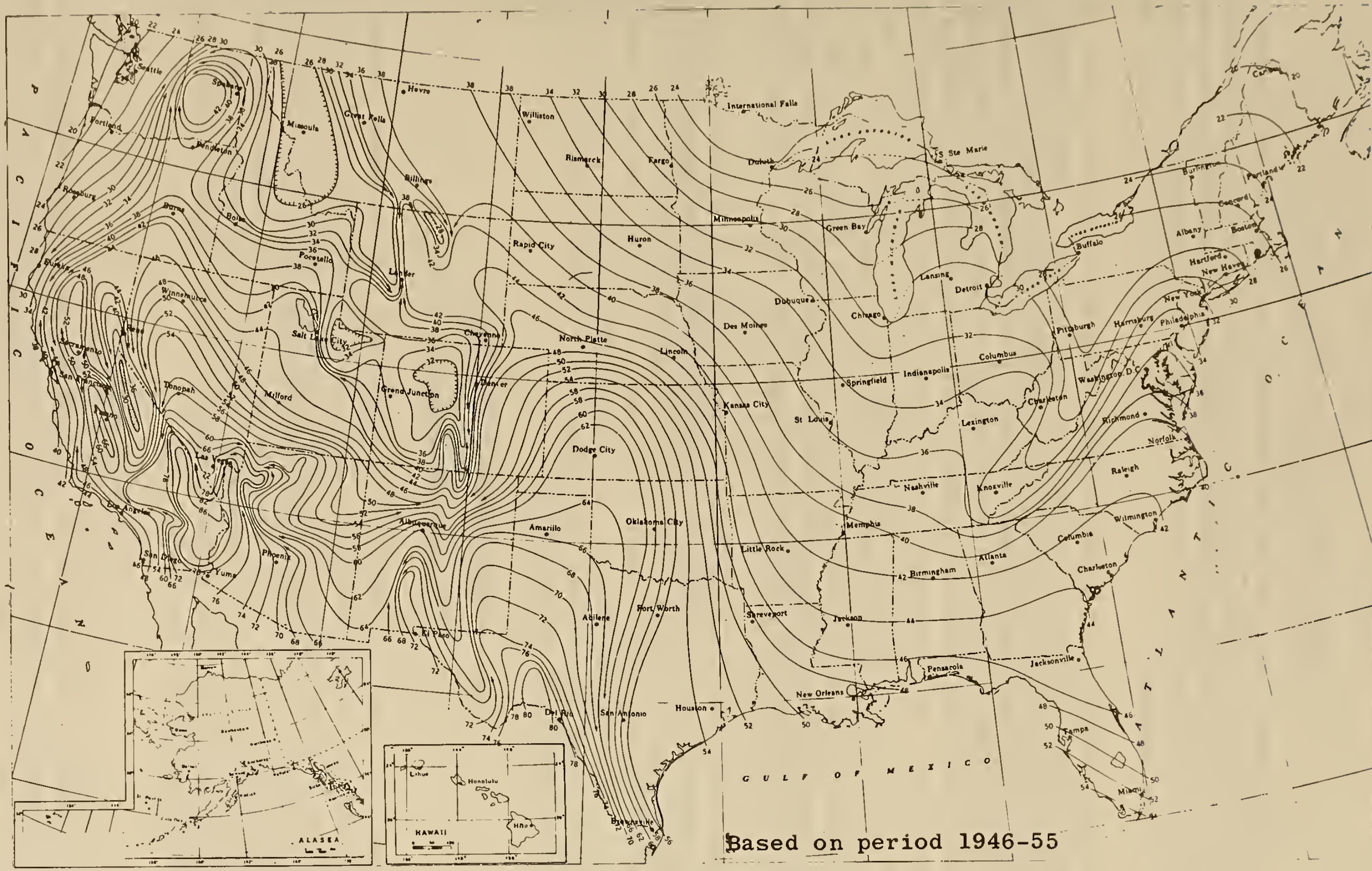



FIGURE A-3. - Mean annual lake evaporation, in inches (25).

# APPENDIX B.--ESTIMATING RUNOFF

## Annual and Seasonal Runoff Volumes

The appropriate design storm for tailings pond design is particularly difficult to determine because of many different disposal facility configurations, sizes, locations, and operational periods. The hydrologic studies may be complex, and consultation with an engineer or hydrologist experienced in this work is recommended.

Predicting seasonal or annual runoff volumes is feasible where the snow accumulates during the winter and melts in the spring. Spring runoff is quite variable from year to year but follows a pattern where long-term precipitation records are available. The 100-year flood is very difficult to predict; however, it is the design parameter that should be considered in tailings pond design unless the pond's location, size, and hazard potential require the use of a larger flood. An unseasonal winter rain and "chinook" winds following a heavy snow can result in catstrophic floods because frozen ground would cause all the rain plus the melted snow to go into the drainage. At lower elevations the entire snow pack could melt and contribute directly to the runoff. At intermediate elevations transition zones exist where the snow absorbs some of the rain. At higher elevations, the snow would absorb all the rain.

In areas where a major portion of the precipitation falls in one season, there will generally be a direct correlation between the annual precipitation and the annual runoff. Snow surveys are a common method of forecasting seasonal runoff from snowmelt. There is a good correlation between snow survey measurements and seasonal runoff. In large drainage areas, however, the runoff varies with the type of precipitation, the degree of saturation of the soil, and the variation in winter melt.

## Flood Runoff From Rainfall

### Maximum Rainfall

The amount of runoff carried by a stream depends on storm characteristics, such as the intensity and duration of the rainfall and the percentage of saturation of the drainage basin. For small drainage basins of several square miles, the rainfall can be assumed to be uniform. An approximation of the quantity of rainfall in a given drainage area can be obtained by comparing precipitation records of other drainage basins in that general area. The National Weather Service has climatic maps showing maximum 24-hour and maximum record storm precipitation values.

### Initial Losses

Initial loss is defined as the maximum amount of precipitation that can occur under specific conditions without producing runoff. In the western mountains and northern plains areas of the United States, a certain amount of infiltration is needed to satisfy the soil deficiencies before runoff occurs. This can vary from a few tenths of an inch up to 2 inches during dry summer

and fall months. If a major rain is preceded by several days of light rain, this runoff lag may not occur. Allowances are generally made for such initial losses. The initial loss in humid regions preceding major floods range from 0.2 to 0.5 inch and is very small as compared to total runoff volume. In the desert country of the Southwest, where flash floods are common, the initial losses in the mountains are virtually nonexistent and the runoff is extremely fast. It is not until the flood waters reach the stream channels that a great deal of infiltration takes place. This can be quite rapid in very permeable soil with a flat stream gradient.

## Infiltration Indices

The infiltration index is the average loss of moisture by seepage into the ground, so all precipitation above this quantity will equal the runoff. Table B-1 is a tabulation of the infiltration indices for several U.S. drainage basins and indicates the vast differences in amount of infiltration in relation to both time and place.

TABLE B-1. - Infiltration indices

| Range, inches per hour | Number of values within various ranges computed from hydrologic records for natural drainage basins | | | | | |
|---|---|---|---|---|---|---|
| | Jan.-Feb. | March-April | May-June | July-Aug. | Sept.-Oct. | Nov.-Dec. |
| NORTHEASTERN U.S. DRAINAGE | | | | | | |
| 0 -0.02 | 0 | 8 | 0 | 0 | 0 | 0 |
| .02- .05 | 0 | 10 | 1 | 0 | 6 | 3 |
| .05- .10 | 0 | 3 | 3 | 0 | 8 | 11 |
| .10- .15 | 0 | 0 | 1 | 0 | 5 | 7 |
| .15- .20 | 0 | 0 | 0 | 0 | 2 | 4 |
| .20- .25 | 0 | 0 | 0 | 0 | 2 | 0 |
| Over .25 | 0 | 0 | 2 | 1 | 6 | 6 |
| Total | 0 | 21 | 7 | 1 | 29 | 31 |
| NORTH U.S. PACIFIC DRAINAGE | | | | | | |
| 0 -0.02 | 3 | 3 | 0 | 0 | 0 | 1 |
| .02- .05 | 4 | 4 | 1 | 0 | 0 | 4 |
| .05- .10 | 2 | 0 | 0 | 0 | 1 | 2 |
| Total | 9 | 7 | 1 | 0 | 1 | 7 |
| ARKANSAS AND RED RIVER BASINS | | | | | | |
| 0 -0.02 | 3 | 1 | 0 | 0 | 0 | 0 |
| .02- .05 | 19 | 12 | 5 | 0 | 3 | 2 |
| .05- .10 | 17 | 33 | 8 | 1 | 1 | 2 |
| .10- .15 | 8 | 21 | 11 | 0 | 4 | 2 |
| .15- .20 | 2 | 14 | 17 | 1 | 3 | 2 |
| .20- .25 | 0 | 6 | 10 | 1 | 0 | 3 |
| Over .25 | 0 | 16 | 46 | 9 | 14 | 0 |
| Total | 49 | 103 | 97 | 12 | 25 | 11 |

Source: U.S. Army Corps of Engineers (47).

## Synthetic Unit Hydrographs

A unit hydrograph represents 1 inch of direct runoff resulting from a rainfall of unit duration and a specific drainage area. The basic premise implies that rainfall of 2 inches will produce a runoff hydrograph having ordinates twice as great as those of the unit hydrograph. The term "unit rainfall duration" refers to the duration of runoff-producing rainfall or rainfall excess that results in a unit hydrograph. The unit hydrograph resulting from a 6-hour unit rainfall duration is referred to as a 6-hour hydrograph. The term "lag" is the length of time from the midpoint of the unit rainfall duration to the peak of the unit hydrograph. A 6-hour unit rainfall duration is suitable and convenient for most studies relating to drainage areas larger than 100 square miles. For drainage areas of less than 100 square miles, values equal to about one-half the lag appear to be satisfactory. Only in approximate studies should unit rainfall durations longer than 12 hours be used because of probable major changes in areal distribution of rainfall during this longer period.

Three methods are available for developing unit hydrographs, as follows:

1. By analysis of rainfall runoff records of isolated unit storms.

2. By analysis of rainfall runoff records for major storms. (These two require rainfall and streamflow records for the actual drainage basin.)

3. By computation of synthetic unit hydrographs from (a) direct analogy with basins of similar charcteristics or (b) indirect analogy with a large number of other basins through the application of empirical relationships.

The basic equations for deriving a synthetic unit hydrograph by this method are shown in equation B-1.

The general procedure is to--

1. Analyze any available hydrologic data available for portions of the drainage area having streamflow records to determine the approximate peak discharge, lag, and general shape of the unit hydrograph.

2. If adequate hydrologic records are available, evaluate coefficients, and use these values in estimating the peak discharge of a synthetic unit hydrograph for the given drainage area. If records are not available, coefficients from adjacent streams with similar characteristics may be used.

3. Evaluate the runoff characteristics involved and estimate whether the unit hydrograph peak discharge values computed for the particular area are consistent with values for comparable basins.

The probable degree of accuracy in the use of unit hydrographs derived from records of minor floods in estimating critical rates of runoff from maximum probable storms indicates that peak discharge ordinates were consistently higher than indicated by the unit hydrographs derived from records of minor

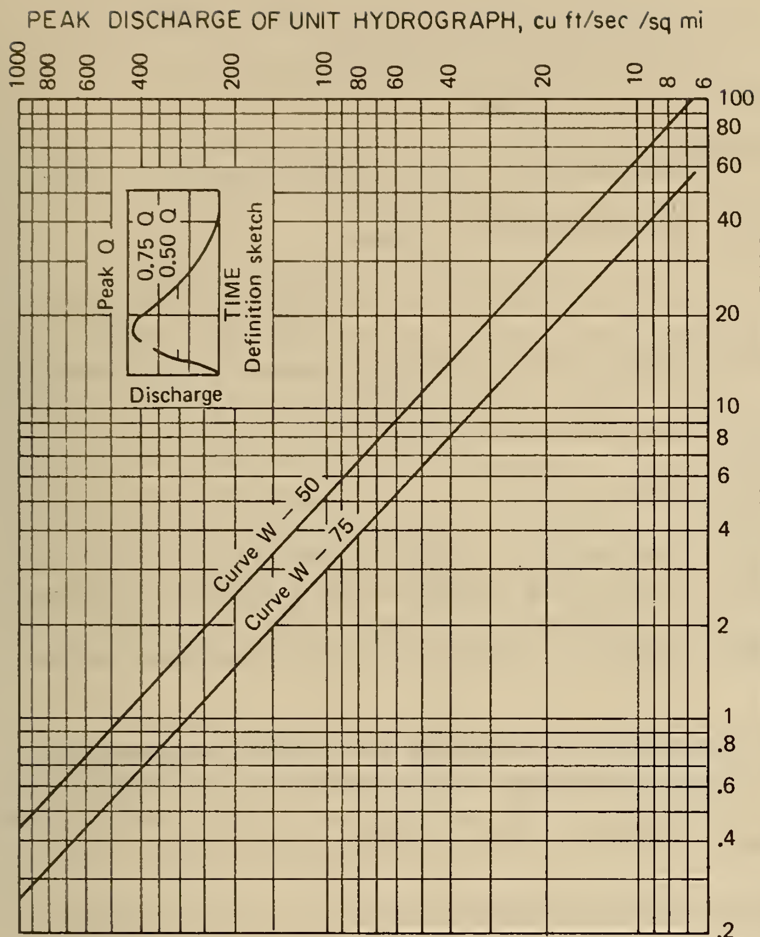



FIGURE B-1. - Unit hydrograph peak versus width (47).

floods in which the areal distribution of rainfall was approximately uniform. In most of the drainage areas considered, the peak ordinates of unit hydrographs derived from major flood hydrographs (<5 inches of runoff) were 25 to 50 percent higher than values computed from records of minor floods (1 to 2 inches). For small drainage areas, if the unit hydrograph has been based on precipitation values that are small in relation to the maximum probable rainfall in the area, the computed peak discharge should be increased by 25 to 50 percent.

It is frequently necessary to modify unit hydrographs derived from available hydrologic records to represent higher rates of runoff. This is done using empirical relationships relating the widths and heights of the peak of the unit hydrographs as shown in figure B-1.

Basic Equations (Source: U.S. Army Corps of Engineers) (47)

$$t_p = C_t \ (LL_{ca})^{0.3}$$

$$t_r = t_p/5.5$$

$$q_p = 640 \ C_p/t_p$$

$$t_{pR} = t_p + 0.25(t_R - t_r)$$

$$q_{pR} = 640 \ C_p/t_{pR} = q_p \ t_p/t_{pR}$$

$$Q_p = q_p \ A \qquad \text{(B-1)}$$

where $t_p$ = lag time of $t_r$ unit hydrograph, hours,

$t_r$ = unit rainfall duration, hours,

$t_R$ = unit rainfall duration other than standard unit $t_r$, hours,

$t_{pR}$ = lag time of $t_R$ unit hydrograph, hours,

$q_p$ = peak discharge rate of the $t_r$ unit hydrograph, cfs/sq mi,

$q_{pR}$ = peak discharge rate of the $t_R$ unit hydrograph, cfs/sq mi,

$Q_p$ = peak discharge rate of $t_r$ unit hydrograph, cfs,

$A$ = drainage area, square miles,

$L_{ca}$ = stream mileage from site to center of gravity of the drainage area (to a point opposite the center of gravity),

$L$ = stream mileage from site to upstream limits of the drainage area,

and $C_t$ and $C_p$ = coefficients depending upon units and basin characteristics. Corresponding values of $C_t$ and 640 $C_p$ follow:

640 $C_p$--range, 200-600; average, 400.

$C_t$--range, 8.0-0.4; average, 2.0

Runoff From Snowmelt (47, 49)

Melting snow affects runoff and may be an important consideration in design flood analyses. Unlike rainfall, snowmelt is not a measured quantity in hydrologic practice, and it must be estimated indirectly from meteorological parameters. Snow hydrology involves primarily the determination of snowmelt rates under various conditions of terrain, vegetation cover, and weather. Secondly, it involves evaluation of the effect of the snowpack on runoff. Thirdly, snow hydrology is concerned with the determination of the water equivalent of the snowpack for use in forecasting total volume of runoff.

Snowmelt During Rain-Free Periods

Evaluating snowmelt on a theoretical basis is a problem of heat transfer involving radiation, convection, and conduction. The effect of these factors in heat transfer is highly variable, depending on the weather and local environment.

The natural sources of heat in melting snow are--

1. Absorbed solar radiation.

2. Net longwave (terrestial) radiation.

3. Convection heat transfer from the air.

4. Latent heat of vaporization by condensation from the air.

5. Conduction of heat from the ground (usually negligible).

6. Heat content of rain water.

Solar radiation is the predominant melting-energy source. Heat transfer to the snow by solar radiation varies with the season, cloud cover, forest cover, latitude, time of day, and reflectivity or albedo of the snow. Since cloud cover affects the amount of radiation that is transmitted to the snow, estimates must be made of the duration of sunshine and of air temperature variations. Methods for estimating this are given in references 47 and 49. A nomograph for estimating solar radiation at latitudes below 50° N is shown in figure B-2. Graphs showing the seasonal and latitudinal variation of solar radiation (in langleys) outside the earth's atmosphere, and seasonal variation of proportional clear sky solar radiation on north and south slopes relative to a horizontal surface are shown in figure B-3.

The albedo is the reflectivity of the snowpack; it varies over a considerable range and is used to measure the amount of solar energy absorbed by the snowpack. It is measured as a percentage of reflected short-wave radiation and ranges from 80 percent for new-fallen snow to as little as 40 percent for melting late-season snow.

Generalized equations have been developed for snowmelt during rain-free periods on the basis of various assumptions and 10 to 80 percent tree cover (equation B-2). The melt coefficients represent the actual melt of the snowpack in inches of water equivalent over the snow-covered area and also express the melt for a ripe snowpack with the isothermal at 30° F and with a 3-percent free water content.

Basic Equations (Source: U.S. Army Corps of Engineers) (49)

Heavily forested area (>80 percent):

$$M = 0.074(0.53\ T_a' + 0.47\ T_d')$$

Forested area (60 to 80 percent):

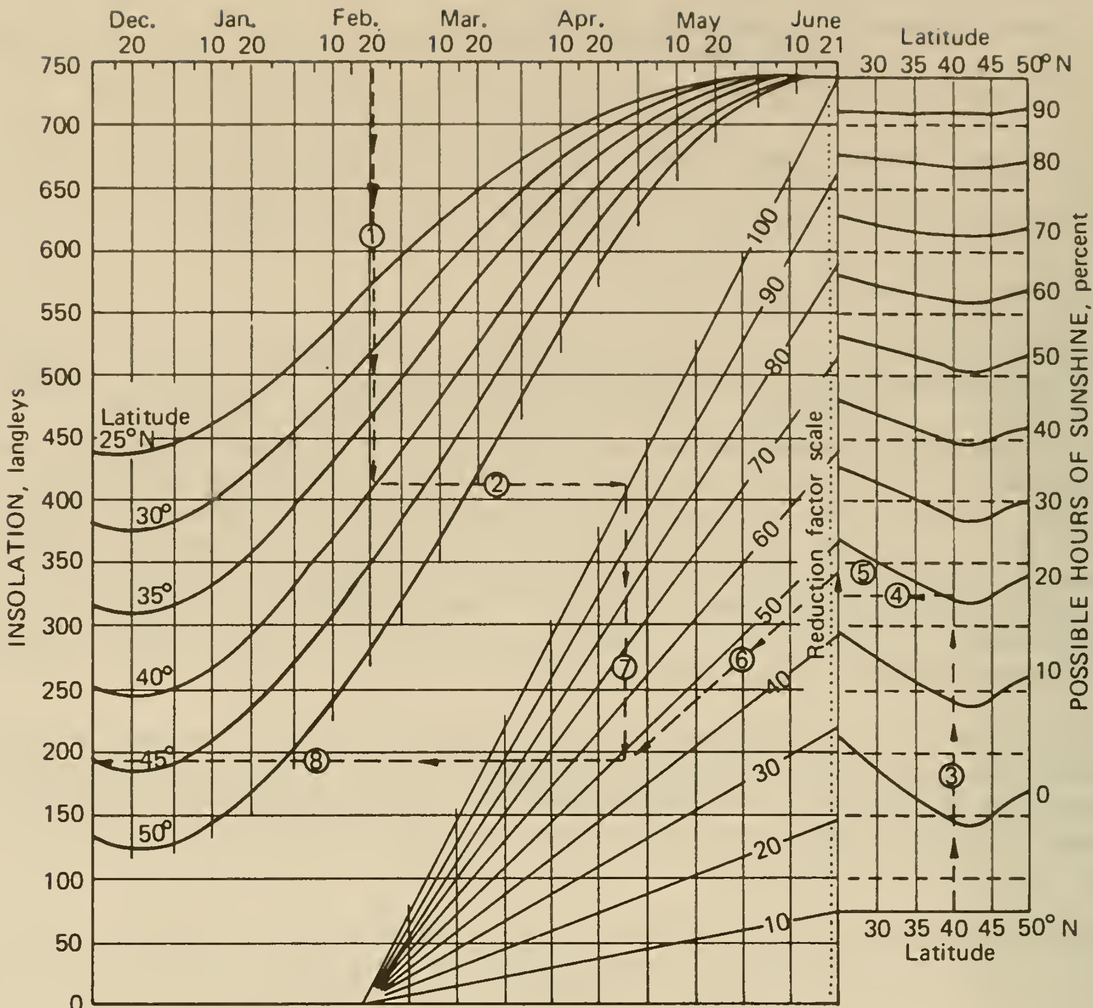
$$M = k(0.0084\upsilon)(0.22\ T_a' + 0.78\ T_d') + 0.029\ T_a'$$

Partly forested area (10 to 60 percent):

$$M = k'(1-F)(0.0040\ I_i)(1-a) + k(0.0084\upsilon)(0.22\ T_a' + 0.78\ T_d') + F(0.0029\ T_a')$$

Open area (<10 percent):

$$M = k'(0.00508\ I_i)(1-a) + (1-N)(0.0212\ T_a' - 0.84) + N(0.029\ T_c') + k(0.0084\upsilon)(0.22\ T_a' + 0.78\ T_d') \qquad \text{(B-2)}$$




Notes:

1. The sample shown by dashed lines estimates the daily total insolation at latitude 40 N on February 21, with 20 percent possible sunshine. Consecutive steps are numbered. Step 5 adds the seasonal correction (+2) read from the table. The final estimate is 195 langleys per day.

2. For use between June 21 and December 21, the curves are symmetrical about June 21.

Seasonal correction to reduction factor

| | Percent of possible sunshine | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Month | 0 | 10 | 20 | 30 | 40 | 50 | 60 | 70 |
| Jan. | +4 | +3 | +3 | +2 | +2 | +2 | +1 | +1 |
| Feb. | +3 | +3 | +2 | +2 | +2 | +1 | +1 | +1 |
| Mar. | −1 | −1 | −1 | −1 | −1 | 0 | 0 | 0 |
| Apr. | −2 | −2 | −1 | −1 | −1 | −1 | −1 | 0 |
| May | −4 | −3 | −3 | −2 | −2 | −2 | −1 | −1 |
| June | −5 | −4 | −4 | −3 | −2 | −2 | −2 | −1 |
| July | −5 | −4 | −3 | −3 | −2 | −2 | −2 | −1 |
| Aug. | −4 | −3 | −3 | −2 | −2 | −2 | −1 | −1 |
| Sept. | −2 | −2 | −1 | −1 | −1 | −1 | −1 | −1 |
| Oct. | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| Nov. | +2 | +2 | +1 | +1 | +1 | +1 | +1 | 0 |
| Dec. | +4 | +3 | +3 | +2 | +2 | +2 | +1 | +1 |

FIGURE B-2. - Solar radiation versus latitude and sunshine (35).

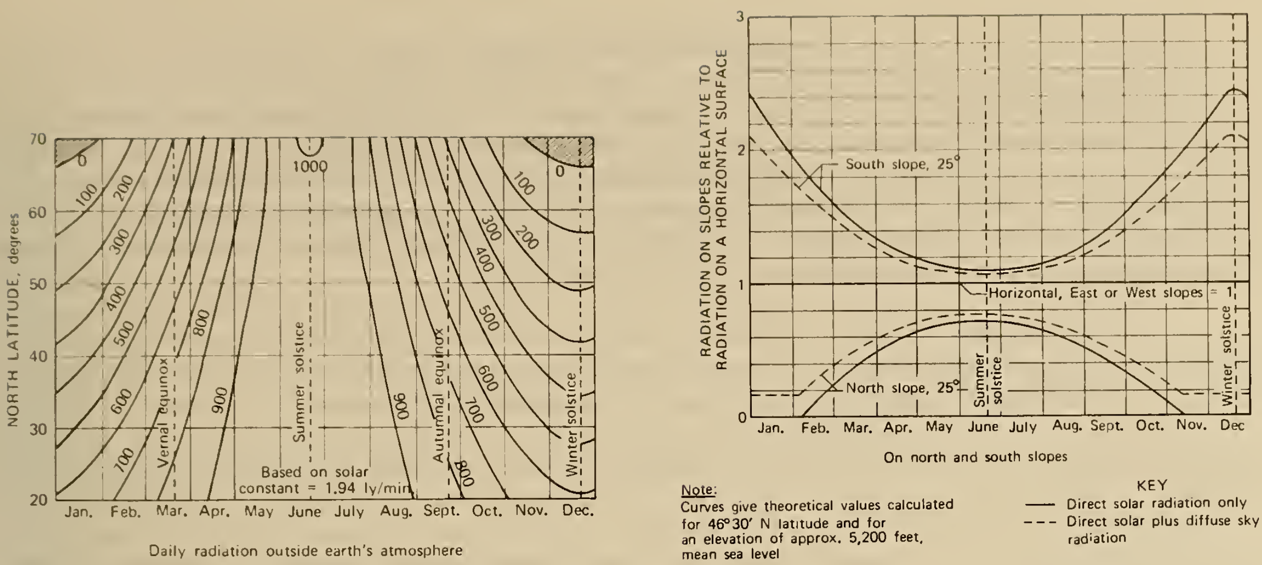



FIGURE B-3. - Clear sky solar radiation (24).

where $M$ = snowmelt rate, inches per day,

$T_a'$ = difference between air and snow surface temperatures, ° F,

$T_d'$ = difference between dewpoint and surface temperatures, ° F,

$\upsilon$ = wind speed in open areas, miles per hour,

$I_i$ = solar radiation on horizontal surface (insolation), langleys,

$a$ = average snow surface albedo,

$k'$ = basin short-wave radiation melt factor (See fig. B-3. It would be 1.0 for a basin essentially horizontal or whose north and south slopes are areally balanced. It usually falls within the limits of 0.9 and 1.1 during spring),

$F$ = average basin forest cover, expressed as a decimal fraction,

$T_c'$ = difference between cloud base and snow surface temperatures, ° F (Air temperature drop 3° to 5° F per 1,000 feet of elevation. Where cloud base is less than 1,000 feet, its temperature can be assumed equal to surface air temperature),

$N$ = cloud cover, expressed as a decimal fraction,

and $k$ = basin convection-condensation melt factor. (See equation B-3.)

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## Snowmelt During Rain

Evaluation of basin snowmelt during rain is a special condition where simplifying assumptions can be made in the snowmelt equation. Solar radiation melt is very small during a rain, but heat transfer by convection and condensation at this time is the major source of energy for snowmelt depending on the type of area. Heat transfer to a snowpack during rain involves the following basic considerations:

1. The amount of heat transfer would depend on the air temperature and velocity of the saturated air. Convection and condensation melt may be active both day and night, especially if there is appreciable wind.

2. This heat transfer is considerably less in the forest than on the plains.

3. Rain melt is simply expressed as a function of rainfall intensity and air temperature.

4. Short-wave radiation melt, ground melt, and evapotranspiration are all negligible. The simplified equations for estimating snow melt during rain follow (equation B-3).

Snowmelt Equations (Source: U.S Army Corps of Engineers) (49)

Heavy forested areas (>80 percent):

$$M = (0.074 + 0.007\ P_r)(T_a - 32) + 0.05$$

Open or partly forested areas (>60 percent):

$$M = (0.029 + 0.0084\ k\upsilon + 0.007\ P_r)(Ta - 32) - 0.09 \qquad \text{(B-3)}$$

where $M$ = snowmelt rate, inches per day,

$Ta$ = mean temperature of saturated air, ° F,

$\upsilon$ = mean wind speed, miles per hour (For partly forested areas, wind values should be those representative of the open portions of the basin),

$P_r$ = rate of rainfall, inches per day,

and $k$ = basin convection-condensation melt factor. (This allows for basin exposure to wind. It would be 1.0 for unforested plains, but could be as low as 0.3 for densely forested areas.)

## Snowmelt and Runoff

Streamflow analysis for winter or early spring requires information on the storage effect of the snowpack. The storage effect of the snowpack on

runoff is determined primarily by the conditions of temperature and liquid water within the pack at any given time. These conditions continually change during the accumulation period so there is no constant amount. During the spring snow-melt period, the snow becomes saturated, will store no more water, and is considered to be primed. After this initial priming, the only additional storage is transitory and there is only a temporary delay of liquid water in transit through the snowpack. This condition is characteristic of the spring period, but it can also occur during midwinter if rainfall can satisfy the cold content and liquid-water-holding capacity of the snowpack.

Equation B-4 is used to calculate the inches of water from either rain or snowmelt necessary to bring a subfreezing snowpack up to 32° F. Added to this must be enough liquid water to saturate the snowpack to 2 to 5 percent of the total water equivalent before runoff occurs. During the day, the snowpack can contain up to 10 percent water equivalent because of water in transit. There is a time delay for runoff to occur, ranging from 3 to 4 hours in mountainous areas to a much longer time on the plains where drainage is not as good.

Cold Content Equation (Source: U.S. Army Corps of Engineers) (49)

$$W_c = \rho\ DTs/160 \qquad \text{(B-4)}$$

where $W_c$ = cold content equivalent, inches of liquid water,

$\rho$ = snow density, grams per cubic centimeter,

$D$ = snowpack depth, inches,

and $Ts$ = average snowpack temperature deficit below 0° C.

## Runoff Losses

The general principles of water loss and delay caused by soil moisture deficits, evapotranspiration, and ground water storage that are used in rain hydrology are also applicable to snow hydrology. There are some differences where snow is a source of runoff. The soil acts as a reservoir, storing water when available. This can be as much as 4 to 8 inches in typical mountain soils. This is sometimes filled to capacity by fall rains prior to the snowfall or early in the snowmelt period. Therefore, essentially no runoff occurs until this deficit is satisfied. A frozen soil mantle will have somewhat the same effect as a saturated soil and will increase the runoff.

Evapotranspiration can account for as much as 12 percent of the water equivalent of the snowpack. Loss by evaporation is very small and will average less than 0.5 inch per month during the winter and early spring.

As much as 30 percent of snowmelt goes into ground water storage, and there is a long time lag before it reaches the streamflow. On the other hand, surface runoff occurs immediately when melting takes place.

Hydrograph synthesis requires a method for evaluating the time-delay runoff for all components of basin storage, including transitory storage in the snowpack, soil, ground water aquifers, and surface-stream channels. Unit hydrographs and storage routing are two methods of evaluating basin storage that are explained in detail elsewhere (47) and will not be discussed further here.

## APPENDIX C.--SEEPAGE AND FLOW NETS (19)

The uppermost line of seepage that is at atmospheric pressure is known as the phreatic surface and is the uppermost flow line.

An equipotential line is a line of equal head; therefore, water rises to the same level in piezometers installed along a given equipotential line. The equipotential lines in a flow net must intersect the free water surface at equal vertical intervals.

Flow lines and equipotential lines must intersect at right angles to form areas that are basically squares when the materials are isotropic. Adjacent equipotentials have equal head losses. The same quantity of seepage flows between adjacent pairs of flow lines.

Usually it is best to start with an integral number of equal potential drops by dividing the total head by a whole number and drawing flow lines to conform to these equal potentials. The outer flow path will generally form a distorted square figure, but the shape of these distorted squares (the ratio B/L) must be a constant (fig. C-1).

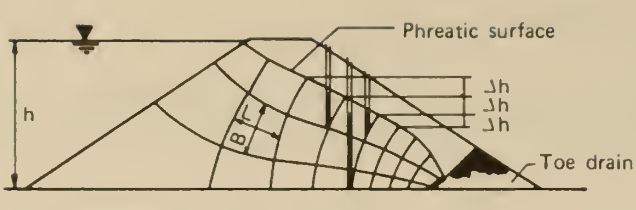
In a stratified soil profile where the ratio of horizontal to vertical permeability ($K_h/K_v$) exceeds 10, the flow in the more permeable layer controls. The flow net may be drawn for the more permeable layer, assuming the less permeable layer is impervious. The head on the interface from this permeable layer is imposed on the less pervious layer for the construction of the flow net within that layer. This situation can occur when a starter dam is impervious to the tailings it is retaining and has insufficient, ineffective, or no drains, so that water at the interface builds up to and goes over the top of the starter dam. The flow net through the starter dam then would be the same as if there was free water against it, as illustrated in figure 49.

In a stratified layer where the ratio of permeability of the layers is less than 10, the flow net is deflected at the interface in accordance with figure C-1.

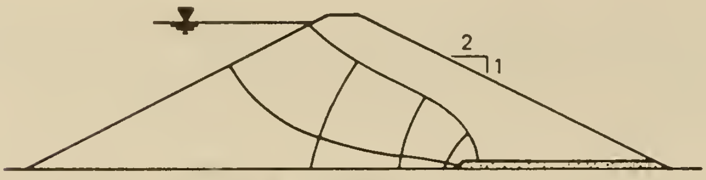
When materials are anisotropic with respect to permeability, the cross section should be transformed by changing the scale, as in figure C-1. The horizontal dimension of the section is reduced by $\sqrt{K_v/K_h}$. The flow net is then drawn as for isotropic materials and can be transposed to a true section. If $K_h > K_v$ the L dimension becomes elongated, and the net is no longer a square. In computing the quantity of seepage, the differential head is not altered for the transformation.

Where only the quantity of seepage is to be determined, an approximate flow net suffices. Where pore pressures are to be determined, the flow net must be accurate.

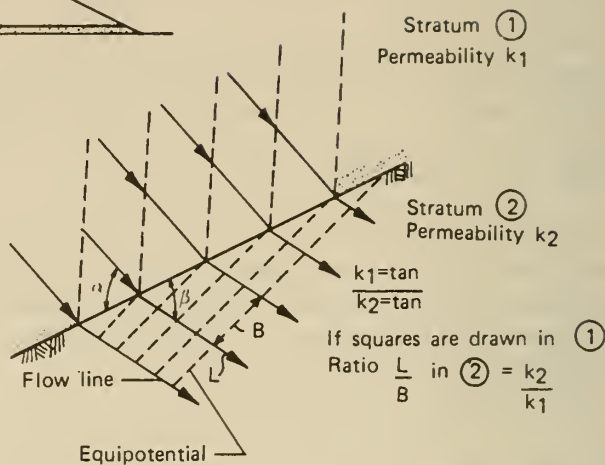
This explanation of flow nets is not intended to make one proficient in drawing flow nets, which requires experience and more detailed explanation.



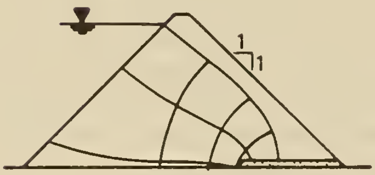

(A) $k_h = k_v$



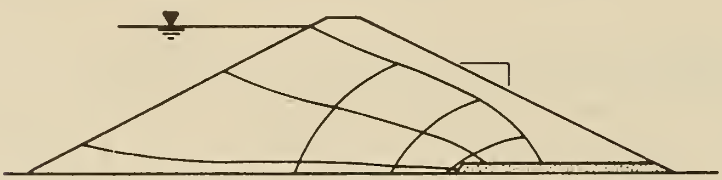

(B) Flow net for true section where $k_h = k_v$




(C) Transfer conditions at interface between strata




(D) Flow net for section (B) above in $k_h = 4k_v$
Horizontal dimensions of section reduced by $\sqrt{\frac{k_v}{k_h}}$



(E) Flow net (D) above transposed to true section

FIGURE C-1. - Flow net construction.

APPENDIX D.--SLOPE STABILITY ANALYSIS, SIMPLIFIED BISHOP METHOD

BISHOP - CASE 1 - HI PHREATIC - DMIN=5.0 - SODERBERG

POINT DATA-- USE 50 POINTS MAXIMUM

| POINT NO. | X-COORD | Y-COORD |
|---|---|---|
| 1 | 1.00 | 200.00 |
| 2 | 200.00 | 200.00 |
| 3 | 275.00 | 230.00 |
| 4 | 320.00 | 205.50 |
| 5 | 339.86 | 255.94 |
| 6 | 650.00 | 380.00 |
| 7 | 850.00 | 378.00 |
| 8 | 850.00 | 313.00 |
| 9 | 850.00 | 293.80 |
| 10 | 850.00 | 229.60 |

LINE DATA---USE 50 LINES MAXIMUM

| | POINT | POINT | SOIL |
|---|---|---|---|
| 1 | 1 | 2 | 3 |
| 2 | 2 | 3 | 3 |
| 3 | 3 | 4 | 3 |
| 4 | 3 | 5 | 2 |
| 5 | 5 | 6 | 1 |
| 6 | 6 | 7 | 1 |
| 7 | 5 | 8 | 2 |
| 8 | 4 | 9 | 2 |
| 9 | 4 | 10 | 3 |

SOIL PROPERTIES--USE 10 SOILS OR LESS

| SOIL NO. | DENSITY PCF | COH. PSI | TAN PHI | PP RATIO | PP RATIO CAPLRY |
|---|---|---|---|---|---|
| 1 | 102.0 | 3.0 | .700 | 1.100 | -0 |
| 2 | 138.0 | 0 | .700 | 1.100 | -0 |
| 3 | 145.0 | 0 | 1.000 | 1.100 | -0 |

PHREATIC SURFACE POINTS--USE 10 POINTS MAXIMUM

| | X-COORD | Y-COORD |
|---|---|---|
| 1 | 1.000 | 200.000 |
| 2 | 200.000 | 200.000 |
| 3 | 339.860 | 255.940 |
| 4 | 850.000 | 313.000 |

THE FOLLOWING IS A PRINTOUT OF THE LINE ARRAY. THE INITIAL 5 LINES
MUST BE THE SURFACE OF THE SLOPE GOING FROM LEFT TO RIGHT.
THERE MUST BE NO VERTICAL LINES AFTER NO. 5 .

| NO. | X-LEFT | Y-LEFT | X-RGHT | Y-RGHT | SLOPE | SOIL |
|---|---|---|---|---|---|---|
| 1 | 1.00 | 200.00 | 200.00 | 200.00 | 0 | 3 |
| 2 | 200.00 | 200.00 | 275.00 | 230.00 | .4000 | 3 |
| 3 | 275.00 | 230.00 | 339.86 | 255.94 | .3999 | 1 |
| 4 | 339.86 | 255.94 | 650.00 | 380.00 | .4000 | 1 |
| 5 | 650.00 | 380.00 | 850.00 | 378.00 | -0.0100 | 1 |
| 6 | 275.00 | 230.00 | 320.00 | 205.50 | -0.5444 | 3 |
| 7 | 339.86 | 255.94 | 850.00 | 313.00 | .1119 | 1 |
| 8 | 320.00 | 205.50 | 850.00 | 293.80 | .1666 | 2 |
| 9 | 320.00 | 205.50 | 850.00 | 229.60 | .0455 | 3 |

NUMBER OF SLICES--100 OR LESS

50.

THE LOWEST ELEVATION THAT SHOULD OCCUR ALONG
ANY TRIAL FAILURE SURFACE (YMIN.)
150.00

THE MINIMUM VALUE FOR THE GREATEST
DEPTH OF THE SLIDING MASS (DMIN).
5.00

1 COMPUTE USING AUTOMATIC SEARCH ROUTINE
2 COMPUTE USING PRESCRIBED CONTROL GRID

1

X AND Y COORDINATES OF THE CENTER OF
THE INITIAL TRIAL FAILURE SURFACE.

X = 269.00 Y = 680.00

INCREMENTS OF X AND Y USED IN THE COARSE GRID
IN SEARCHING FOR THE MINIMUM FACTOR OF SAFETY.
THE FINAL GRID IS 4 TIMES FINER.

X = 20.000 Y = 20.000

| X COORD. | Y COORD. | RADIUS | FS BISHOP | FS FELLINIUS |
|---|---|---|---|---|
| 269.00 | 680.00 | 462.92 | 1.687 | 1.644 |
| 289.00 | 680.00 | 463.26 | 1.711 | 1.652 |
| 249.00 | 680.00 | 471.71 | 1.665 | 1.616 |
| 229.00 | 680.00 | 475.93 | 1.642 | 1.598 |
| 209.00 | 680.00 | 477.19 | 1.618 | 1.586 |
| 189.00 | 680.00 | 475.40 | 1.578 | 1.565 |
| 169.00 | 680.00 | 478.28 | 1.551 | 1.545 |
| 149.00 | 680.00 | 480.09 | 1.518 | 1.520 |
| 129.00 | 680.00 | 473.99 | 1.231 | 1.240 |
| 109.00 | 680.00 | 483.57 | 1.227 | 1.231 |
| 89.00 | 680.00 | 488.00 | 1.248 | 1.254 |
| 109.00 | 700.00 | 503.86 | 1.300 | 1.306 |
| 109.00 | 660.00 | 465.14 | 1.170 | 1.170 |
| 109.00 | 640.00 | 446.45 | 1.215 | 1.215 |
| 129.00 | 660.00 | 455.36 | 1.008 | 1.008 |
| 149.00 | 660.00 | 449.74 | 1.486 | 1.506 |
| 129.00 | 680.00 | 473.99 | 1.231 | 1.240 |
| 129.00 | 640.00 | 438.67 | 1.182 | 1.185 |
| 134.00 | 660.00 | 453.76 | 1.132 | 1.137 |
| 124.00 | 660.00 | 458.69 | 1.196 | 1.201 |
| 129.00 | 665.00 | 460.02 | .999 | .998 |
| 129.00 | 670.00 | 464.67 | .997 | .996 |
| 129.00 | 675.00 | 469.33 | 1.113 | 1.118 |
| 134.00 | 670.00 | 463.07 | 1.332 | 1.345 |
| 124.00 | 670.00 | 470.24 | 1.282 | 1.287 |

MINIMUM BISHOP FS FOUND AT X= 129.00, Y= 670.00,
R= 464.67, BSHP.= .997, FELL.= .996

BISHOP - CASE 2 - LO PHREATIC - DMIN=5.0 - SODERBERG

POINT DATA-- USE 50 POINTS MAXIMUM

| POINT NO. | X-COORD | Y-COORD |
|---|---|---|
| 1 | 1.00 | 200.00 |
| 2 | 200.00 | 200.00 |
| 3 | 275.00 | 230.00 |
| 4 | 320.00 | 205.50 |
| 5 | 339.86 | 255.94 |
| 6 | 650.00 | 380.00 |
| 7 | 850.00 | 378.00 |
| 8 | 850.00 | 313.00 |
| 9 | 850.00 | 293.80 |
| 10 | 850.00 | 229.60 |

LINE DATA---USE 50 LINES MAXIMUM

| | POINT | POINT | SOIL |
|---|---|---|---|
| 1 | 1 | 2 | 3 |
| 2 | 2 | 3 | 3 |
| 3 | 3 | 4 | 3 |
| 4 | 3 | 5 | 1 |
| 5 | 5 | 6 | 1 |
| 6 | 6 | 7 | 1 |
| 7 | 5 | 8 | 1 |
| 8 | 4 | 9 | 2 |
| 9 | 4 | 10 | 3 |

SOIL PROPERTIES--USE 10 SOILS OR LESS

| SOIL NO. | DENSITY PCF | COH. PSI | TAN PHI | PP RATIO | PP RATIO CAPLRY |
|---|---|---|---|---|---|
| 1 | 102.0 | 3.0 | .700 | 1.100 | -0 |
| 2 | 138.0 | 0 | .700 | 1.100 | -0 |
| 3 | 145.0 | 0 | 1.000 | 1.100 | -0 |

PHREATIC SURFACE POINTS--USE 10 POINTS MAXIMUM

| | X-COORD | Y-COORD |
|---|---|---|
| 1 | 1.000 | 200.000 |
| 2 | 200.000 | 200.000 |
| 3 | 320.000 | 205.500 |
| 4 | 850.000 | 293.800 |

THE FOLLOWING IS A PRINTOUT OF THE LINE ARRAY. THE INITIAL 5 LINES MUST BE THE SURFACE OF THE SLOPE GOING FROM LEFT TO RIGHT. THERE MUST BE NO VERTICAL LINES AFTER NO. 5 .

| NO. | X-LEFT | Y-LEFT | X-RGHT | Y-RGHT | SLOPE | SOIL |
|---|---|---|---|---|---|---|
| 1 | 1.00 | 200.00 | 200.00 | 200.00 | 0 | 3 |
| 2 | 200.00 | 200.00 | 275.00 | 230.00 | .4000 | 3 |
| 3 | 275.00 | 230.00 | 339.86 | 255.94 | .3999 | 2 |
| 4 | 339.86 | 255.94 | 650.00 | 380.00 | .4000 | 1 |
| 5 | 650.00 | 380.00 | 850.00 | 378.00 | -0.0100 | 1 |
| 6 | 275.00 | 230.00 | 320.00 | 205.50 | -0.5444 | 3 |
| 7 | 339.86 | 255.94 | 850.00 | 313.00 | .1119 | 2 |
| 8 | 320.00 | 205.50 | 850.00 | 293.80 | .1666 | 2 |
| 9 | 320.00 | 205.50 | 850.00 | 229.60 | .0455 | 3 |

NUMBER OF SLICES--100 OR LESS

50.

THE LOWEST ELEVATION THAT SHOULD OCCUR ALONG ANY TRIAL FAILURE SURFACE (YMIN.)

150.00

THE MINIMUM VALUE FOR THE GREATEST DEPTH OF THE SLIDING MASS (DMIN).

5.00

1 COMPUTE USING AUTOMATIC SEARCH ROUTINE
2 COMPUTE USING PRESCRIBED CONTROL GRID

1

X AND Y COORDINATES OF THE CENTER OF THE INITIAL TRIAL FAILURE SURFACE.

X = 269.00 Y = 680.00

INCREMENTS OF X AND Y USED IN THE COARSE GRID IN SEARCHING FOR THE MINIMUM FACTOR OF SAFETY. THE FINAL GRID IS 4 TIMES FINER.

X = 20.000 Y = 20.000

| X COORD. | Y COORD. | RADIUS | FS BISHOP | FS FELLINIUS |
|---|---|---|---|---|
| 269.00 | 680.00 | 462.92 | 2.361 | 2.281 |
| 289.00 | 680.00 | 458.39 | 2.325 | 2.241 |
| 309.00 | 680.00 | 456.45 | 2.300 | 2.205 |
| 329.00 | 680.00 | 457.56 | 2.309 | 2.199 |
| 309.00 | 700.00 | 475.61 | 2.289 | 2.197 |
| 309.00 | 720.00 | 497.41 | 2.292 | 2.199 |
| 329.00 | 700.00 | 468.46 | 2.317 | 2.223 |
| 289.00 | 700.00 | 477.52 | 2.311 | 2.230 |
| 314.00 | 700.00 | 477.17 | 2.294 | 2.196 |
| 304.00 | 700.00 | 476.70 | 2.292 | 2.201 |
| 309.00 | 705.00 | 480.40 | 2.287 | 2.197 |
| 309.00 | 710.00 | 487.82 | 2.292 | 2.197 |
| 314.00 | 705.00 | 481.97 | 2.293 | 2.196 |
| 304.00 | 705.00 | 481.49 | 2.289 | 2.200 |

MINIMUM BISHOP FS FOUND AT X= 309.00, Y= 705.00,
R= 480.40, BSHP.= 2.287, FELL.= 2.197

## APPENDIX E.--NOMENCLATURE

Borrow material.--Natural soil, rock, or manmade waste (mill tailings) obtained from outside the construction site and used for making fills, dams, roads, etc.

Bulking.--A term used to describe a moist sand the particles of which are held in a loose structure by capillary forces between individual grains. Either drying or saturation will release the capillary forces and allow the particles to shift into a denser structure.

Cohesion (c).--A strength exhibited by some soils even though there is no normal stress applied to the soil. The exact cause of cohesion is unknown; however, it is felt to be related to intergranular attraction and capillarity of partially saturated soil.

Decant.--A decant tower in a tailings pond controls the water level of the pond. It collects the clear water and directs it to a holding pond outside the downstream toe of the embankment. Figures 18, 19, and 21 are decant tower and line pictures.

Driving force.--The weight of soil and water and inertia which may cause sliding along the potential failure surface. See also Resisting force.

Dynamic factor of safety.--Factor of safety calculated as affected by seismic or other outside dynamic force. See also Static factor of safety.

Factor of safety (FS).--The factor of safety for stability of an earth mass is the summation of resisting forces divided by the summation of driving forces tending to cause movement. An FS figure of 1.0 or less indicates potential failure, and above 1.0 is stable. An FS of 1.5 is generally considered necessary for a safe tailings pond.

Leaching.--The process by which water (or water and sulfuric acid) is sprayed or injected into "waste" dumps containing copper mineralization too low in grade to be milled. The resulting copper sulfate solutions are run through vats containing scrap iron, where the copper is precipitated for shipment to the smelter.

Maximum storm.--Amount of precipitation in any given storm, regardless of length of storm.

Permeability.--This is the measure of the rate of flow of water through a given soil--generally measured in centimeters per second. $1 \times 10^{-6}$ centimeters per second is approximately 1 foot per year.

Constant-head permeability is the simplest type of test; the waterhead on the soil is maintained by using a fixed reservoir with a constant level of water above the soil sample.

Falling head permeability is a test in which the upper reservoir is replaced with a vertical stand pipe. During the test the level of water falls, and the flow is measured by the difference in volume in the standpipe.

Phi angle (ϕ).--The angle of internal friction between soil particles. In addition to the friction between minerals, it includes interlocking between individual particles. It is measured by direct shear test or triaxial shear test measured in degrees and is not affected by water.

Phreatic surface.--The line below which the soil is saturated. It can also be called the ground water line or the top flow line in a flow net.

Resisting force.--Shearing resistance of the soil mass and other forces that oppose the driving force.

Shear box.--In the direct shear test a sample of soil is placed in a container the top half of which can slide freely over the bottom half. A normal load is applied to the top of the sample, and a shearing force is applied to the top half of the container, shearing the sample horizontally at the midpoint.

Shear strength (peak).--The maximum shearing stress achieved during a shear test with a given normal load is designated as the peak strength.

Shear strength (residual).--After the shear stress has peaked, if the test is continued, an ultimate or residual stress will be attained. It is usually constant and less than the peak strength.

Stability analysis.--One of several methods of determining the safety of an earth mass against failure or movement.

Static factor of safety.--Factor of safety calculated with no dynamic forces acting on the embankment.

Tailings.--The U.S. Department of the Interior, Bureau of Mines, dictionary of mining, mineral, and related terms requires its use rather than the word tailing which is preferred by metallurgists.

Time of concentration.--Length of time when precipitation is intense in any given storm.

U.S. GOVERNMENT PRINTING OFFICE : 1977 O—247-978